EFFICIENCY OF BIORETENTION CELLS FOR STORMWATER MANAGEMENT WATER QUANTITY CONTROL FOR LOW DENSITY RESIDENTIAL DEVELOPMENT: A CASE STUDY IN THE GREATER TORONTO AREA

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Abstract

Dealing with stormwater runoff from water quantity and water quality aspects has always been a category of necessary municipal infrastructure. Bioretention Cells (BRCs) are new features of this infrastructure. A BRC is a basin in the ground that consists of a surface bowl to initially capture runoff which can filter through a permeable media. The primary objective of BRCs is for water quality treatment but their contribution to water quantity control objectives of an overall stormwater design for a medium density subdivision is not fully understood.

From 2011 to 2017, a low-density residential subdivision in the greater Toronto Area, Mosaik Glenway, was constructed as a pilot study. Its stormwater system includes three BRCs. The objective of this research is to use the results of monitoring and computer modelling of the Mosaik Glenway subdivision to help understand the contribution that BRCs can make to quantity control in stormwater management (SWM).

Modelling results presented in this thesis demonstrate that BRCs can help mitigate the loss of infiltration due to urbanisation. Under favourable soil conditions, they may have the potential to fully mitigate the loss of infiltration due to urbanization. BRCs are also capable of reducing the amount of quantity storage normally required by traditional SWM facilities for a wide range of precipitation events, including large storm events, even if they are constructed in less-than-favourable soil conditions. Moreover, BRCs also have opportunities to reduce overall stormwater management infrastructure costs.

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List of Abbreviations and Notations

A	area (ha) or (m^2)
BAV	bioretention abstraction volume
BMP	best management practices
BRC	bioretention cell
C	runoff co-efficient
CHI	Computational Hydraulics International
CVC	Credit Valley Conservation
DCI	directly connected impervious
$d_{p75-p25}$	depth from 75% full to 25% full
ECA	Environmental Compliance Approval (issued by MECP)
EPA	Environment Protection Agency
ET	evapotranspiration
f	infiltration rate (mm/hr)
GWT	groundwater table
i	rainfall intensity (mm/hr)
I	infiltration
IC	indirectly connected
IWS	internal water storage
K	saturated hydraulic conductivity
LSRCA	Lake Simcoe Region Conservation Authority
LID	low impact development
MECP	Ministry of Environment, Conservation and Parks (of Ontario)
MOE	Ministry of Environment (predecessor to MECP)
P	precipitation
PCSWMM	Personal Computer Stormwater Management Model (by CHI)

Q_p	peak flow (m^3/s)
R	runoff
SCS	Soil Conservation Service
STEP	Sustainable Technologies Evaluation Program
SWMPD	Stormwater Management Planning and Design
SWM	stormwater management
SWMM	Stormwater Management Model (by EPA in the US)
t_c	time of concentration (min)
$t_{p75\text{-}p25}$	time from 75% full to 25% full
TRCA	Toronto Region Conservation Authority
TSS	total suspended solids
TTT	Treatment Train Tool
$V_{p75\text{-}p25}$	volume between 75% full to 25% full

List of Symbols

Ø	diameter

Chapter 1: Introduction

1.1 Background

Managing urban stormwater runoff, especially from major storm events, has always been a category of necessary municipal infrastructure for an urban area to function. Floods resulting from stormwater can have catastrophic economic and social consequences. Prior to the 1970`s, the central tenet was to convey stormwater away from urban areas by concrete infrastructure to receiving water bodies as quickly as possible. Unfortunately, such practices created erosion damage at receiving water bodies and also contributed to degradation of receiving water bodies (water quality).

Since the late1970’s, stormwater management (SWM) evolved (TRCA & CVC, 2010). Traditional SWM practices call for construction of SWM ponds located at the downstream ends of concrete conveyance infrastructure. Recent studies have shown SWM ponds to fall short of some of their objectives, especially in mitigating the harmful effects of stormwater on water quality (LSRCA, 2011).

The latest trend is to replicate as much as possible slower natural hydrological processes, such as infiltration and evapotranspiration, throughout a site or region, rather than quick conveyance by concrete infrastructure to an “end-of-pipe” facility. This new trend has been labelled Low Impact Development (LID), Sustainable Urban Drainage Systems, amongst other titles. It consists of an array of techniques to be used solely or in combinations as part of a treatment train. One of the techniques is known as bioretention.

A Bioretention Cell (BRC) is a vegetated basin in the ground that consists of a surface bowl to initially capture runoff which can infiltrate to the subsurface through a permeable growing media. Besides filtering runoff, this media also promotes the growth of vegetation. In many cases water then will have an opportunity to infiltrate into the ground and/or be conveyed by underground pipes to a receiving water body. While the runoff filters through the permeable media of the BRC it is subjected to many physical, chemical, and biological processes that treats the runoff for water quality whilst reducing the peak flow and volume of runoff (Khan et al, 2012(part 1 and 2)).

BRCs have been the subject of many research studies to better understand their efficacy in managing urban stormwater runoff. For example, BRCs have been studied under laboratory conditions (Li et al, 2008) (Gulbaz and Kazezyilmaz-Alhan, 2016) and in field applications with controlled water input (Khan et al, 2012). However, there are few cases where BRCs have been studied in detail to determine what extent they can help fulfill design requirements for an entire medium density residential subdivision. Zimmer et al (Zimmer, 2007) attempted to address this question in 2007, however their approach has various limitations, especially addressing infiltration. More is now known in 2021 and modelling techniques present more options to quantify the performance of LID facilities.

From 2011 to 2017, a low-density residential subdivision in the Greater Toronto Area, Mosaik Glenway, was designed and constructed as a pilot study with the application of LID techniques, including BRCs. This pilot study is also being monitored, especially in regard to infiltration. This project provides an opportunity to study how BRCs support the many water quantity facets in the overall design of stormwater management infrastructure of an urban development.

1.2 Thesis Objective

Lake Simcoe Region Conservation Authority (LSRCA) has attempted to quantify the contribution that BRCs can make to the quantity control aspects of a SWM infrastructure system by relating those quantifications to the infiltration rate of the soil underlying the BRC. If a BRC is constructed in a suitable soil that meets their criteria and also other conditions, then a control volume credit equivalent to the quantity of runoff from a 25 mm rainfall event from a proponent’s site is given (LSRCA, 2016). The objective of this research is to use numeric modelling and the results of monitoring of the recently constructed Mosaik Glenway subdivision to help understand the contribution that BRCs can make to quantity control in stormwater management. Modelling will be done based on continuous precipitation data and on design storm events to measure various performance differences. Simulations will be done for pre-development conditions, post-development conditions without any mitigation and post-development conditions with SWM mitigation, including BRCs. For many of these scenarios, a design process will be executed to determine stormwater storage quantities. Using the outcome from this research, academics, professionals and conservation agencies will have more evidence in quantifying the contributions that BRCs can make to the quantity control aspects of a SWM infrastructure system and apply an appropriate control volume credit, which may be greater than or less than the quantity of runoff from a 25 mm rainfall event from a proponent's site.

Since LID techniques are relatively new, there are uncertainties about corresponding capital and long-term maintenance costs (STEP, 2013). Therefore, besides evaluation of technical hydrological efficiency of BRCs, this thesis will also include an economic component as it will compare the capital and maintenance costs of a system utilizing traditional SWM facilities versus a system designed using BRCs. In both cases, the systems are to meet the same design approval criteria.

1.3 Thesis Layout

This thesis has five chapters, as follows:

Chapter 1 (Introduction): Outlines background information, thesis objective and layout.

Chapter 2 (Relevant Literature): Presents the development of BRCs along with design and performance aspects.

Chapter 3 (Methodology): Provides details of the Mosaik Glenway site, monitoring, preparation for computer modelling and cost examination.

Chapter 4 (Results and Discussion): Presents the results of monitoring, computer model results using continuous precipitation data, modeling with design storm events and cost examination.

Chapter 5 (Conclusion and Recommendations): Outlines the conclusions and recommendations for future study.

Chapter 2: Relevant Literature

2.1 Stormwater Management

Prior to the concept of SWM in the 1980's, stormwater was conveyed directly by storm sewers or hard surfaced channels to receiving bodies of water with resulting destructive forces on the environment. Erosive damage and downstream sediment deposition were due to increased peak flow and increased runoff volume resulting from increased imperviousness and reduced times of concentration of flow. To solve the problem, runoff quantity control was implemented, in the form of dry SWM detention ponds to decrease peak flows of design storms representing 2-, 5-, 10-, 25-, 50- and 100-year return periods. Dry SWM detention ponds for major storm events greater than 5-year return periods were often incorporated in recreational fields, such as soccer pitches or baseball diamonds, within residential communities, to decrease peak flows by employing temporary impoundment to delay the flow of water over a longer period. The assumption was that such recreation fields would not be used during major storm events.

In the 1980's, the concept of minor and major drainage systems was introduced. The minor system is the underground pipe conveyance system (commonly known as storm sewers) designed to convey up to 2-to-10-year storm events, depending on local municipality design criteria. The major system utilizes paved rights-of-way when the pipe conveyance system capacity is exceeded and conveys flows from major storm events, designed for up to the 100-year storm, safely to a free surface waterbody, such as a watercourse or lake.

Subsequently, concerns evolved regarding water quality due to rainfall runoff passing over paved surfaces which picked up deposition from the pavement, such as vehicle wear materials from engines, tires, and brakes (Davis et al, 2009). These materials, as well as hard materials from erosion are known as Total Suspended Solids (TSS). Nutrients, such as nitrogen and phosphorus, are naturally occurring chemical elements in our environment and can be components of fertilizers; however, when they are not consumed by vegetation or absorbed into the soil via natural processes, they accumulate and are conveyed to receiving bodies of water via paved surfaces and pipes. The excess nutrients can cause reactive processes, such as eutrophication, in receiving bodies of water. All the above are considered water pollutants.

In response to the impact of these water pollutants, wet SWM ponds evolved to also include permanent pools of water for water quality treatment. Eventually, the hydraulic design of SWM ponds were modified to also treat smaller events, such as the 25 mm precipitation event. Higher levels of imperviousness from urbanization create higher volumes of water for all storms and therefore there is more erosive action. As per requirements of agencies, such as Toronto and Region Conservation Authority (TRCA), the volume of the 25 mm event is to be released over 24 or 48 hours for this type of control. In some cases, even more stringent runoff criteria from fluvial geomorphological assessments are used.

However, according to recent studies (LSRCA, 2011) modern SWM ponds are falling short of sufficiently mitigating the damaging effects of stormwater on water quality. They also do not address other detrimental impacts on the environment caused by urban development, such as reduced infiltration which affects groundwater recharge and reduced baseflow in streams. In addition, reduced evapotranspiration, increased water temperature, reduced depression storage and reduced interception by vegetation created by urbanization are not mitigated by SWM ponds. It has also been noted that some municipalities are not fulfilling the required maintenance on SWM ponds (LSRCA, 2011). SWM ponds need to periodically have the accumulated sediment removed from them in order to maintain their intended function.

Regardless of the shortcomings of SWM ponds to deal with some of the above quality and hydrological aspects, they still have value in detaining volumes of water of frequent events, such as the 25 mm event, and reducing peak flows of storm events, up to the 100-year event. In some cases, conservation authorities in Southern Ontario require storage in SWM ponds to control peak flows from the regional event, which in Southern Ontario, is Hurricane Hazel (TRCA, 2012).

The above discussion can be summarized by the following table adapted from a chronology and other developments of stormwater management graphically represented in the *LID Planning and Design Guide* (TRCA & CVC, 2010):

Table 2.1: Summary of the Evolution of Stormwater Management

Time Period	**Additional Stormwater Practices Introduced during Time Period**
1980’s	Floodplain Management
	Runoff Quantity Control
	Erosion /Flood Control
	Minor & Major Conveyance System
1990’s	Baseflow Maintenance
	Fisheries/Aquatic Habitat
	Water Quality
2000’s	Fluvial Geomorphology
	Monitoring
	Enhancement Opportunities
	Groundwater Infiltration
	Water Temperature
2010’s	Climate Change
	Water Balance
	Low Impact Development

Currently, any or all the items listed above in Table 2.1, may have to be addressed in mitigating impacts of urban development on the environment. As a result of some of the shortcoming of the status quo of SWM ponds, discussed above, the concept of LID has recently become of great interest to water resources engineers to address issues of SWM.

2.2 Low Impact Development

This latest development in stormwater management, LID, employs various techniques to replicate as closely as possible pre-development hydrological characteristics. Besides changes in planning, LID techniques attempt to distribute the treatment of stormwater over the tributary area, rather than concentrated end-of-pipe treatment, such as a SWM pond. The *LID Planning and Design Guide* (TRCA & CVC, 2010) categorizes LID techniques as follows:

1. Rainwater Harvesting, such as rain barrels or cisterns.
2. Green Roofs constructed on flat roofs.
3. Roof Downspout Disconnection, as compared to downspout directly connected to storm sewers.
4. Soakaways, Trenches and Chambers for infiltration purposes.
5. Bioretention, also known as Bioretention Cells or Rain Gardens, including Biofilters.
6. Vegetated Filter Strips mainly associated with highways.
7. Permeable Pavement (permeable pavers, permeable asphalt and permeable concrete).
8. Enhanced Grass Swales, including roadside ditches that have check dams, or similar measures to reduce runoff function.
9. Dry Swales, a version of Enhanced Grass Swales that have a perforated pipe underneath them.
10. Perforated Pipe Systems, including perforated storm sewers under roads.

The above measures can be used individually, or in combinations, to create treatment trains. Treatment trains can also include large detention facilities, such as underground vaults or dry detention ponds, to handle large infrequent storm events that would otherwise be too large for a LID facility to control (Davis et al, 2012).

To help promote LID facilities, an organization named Sustainable Technologies Evaluation Program (STEP) was created. It is a collaborative effort of three conservation authorities in Southern Ontario: TRCA, Credit Valley Conservation (CVC), and LSRCA.

STEP publishes many studies, especially regarding projects that use LIDs, to help facilitate the development of LID techniques.

2.3 Bioretention Cells

2.3.1 Introduction

BRCs are one of the most promising LID methods. BRCs utilize knowledge from many fields including soil science, horticulture, engineering hydrology and hydraulics, hydrogeology, and landscape architecture (Davis et al, 2009). Physical impacts of BRCs can be wide ranging, from hydraulic quantity performance on peak flow, volume and time of concentration to maintenance of groundwater levels and base flow in watercourses (Davis et al, 2009). BRCs also promote evapotranspiration. While this research primarily focuses on the water quantity aspects of BRCs, they have many water quality treatment attributes. The impact of BRCs on water quality in receiving water bodies can be achieved through multiple treatment processes that may include many of the following (Davis et al, 2009):

- Retention (permanent abstraction from surface flow)
- Ground Infiltration
- Filtration
- Precipitation
- Biological activity
- Phytoremediation
- Chemical sorption
- Heat transfer

For the first two process in the above list, the contribution to water quality in receiving water bodies is indirect. These processes result in the water not reaching the receiving water body but instead being captured and re-directed.

2.3.2 Types of Bioretention Cells

Although BRCs may look alike from the surface, they can be quite varied based on their intended usage. BRCs were initially conceived as mainly filtration devices (Davis et al, 2009). Some may have indeed been used for filtration reasons, while others may have also been used for infiltration into the subsurface, or both. It should be noted that the water that does not infiltrate into the ground, may still exit a BRC having received the benefit of filtration. Whether a BRC is used solely as a filtration device, or in combination with infiltration, depends on geotechnical and/or hydro-geotechnical matters. If the surrounding soils are cohesive type soils, such as silt or clay, only filtration purposes should be considered; whereas, if the surrounding soils are non-cohesive, such as sand or gravel, then sub-surface infiltration can play a large role. To employ infiltration, the surrounding soil must have an infiltration rate of at least 1.3 cm/hr (Davis et al, 2009). There should be at least 1 m separation from the bottom of the BRC to the groundwater table (GWT) for water quality treatment to occur (MOE, 2003). If the bottom to the BRC is too close to the GWT, then there is the danger of groundwater discharge to the surface via the BRC, especially if it has an underdrain pipe. If a GWT exists less than 1 m from the bottom of the BRC, generally, the BRC should be encased with an impermeable liner so there is no interaction with the GWT. The same would also apply to a site where there is a potential danger of a contaminant entering the GWT from an industrial land use at the surface.

2.3.3 Bioretention Cell Design

2.3.3.1 Physical Components

There are many design features to consider in the design of a BRC, as follows (Davis et al, 2009):

- Maximum pooling depth
- Minimum filter media depth
- Filter media composition and configuration
- Underdrain (if applicable) configuration
- Pre-treatment options, especially in commercial and industrial areas
- Vegetation selection
- Surface area and corresponding volume
- Overflow design
- Maintenance/ service life/ inspection

The interaction between physical components of a BRC are extremely important (See Figure 2.1) (Davis et al, 2012).

		↑ *Evapotranspiration*	
Inflow/Pollutants →	Upper Bowl Surface Storage (0.3 m +/-)		→ *Bowl Overflow*
		↓ *Infiltration*	
	Filter Media Storage (1 m +/-)		→ *Underdrain Discharge*
		↓ *Percolation*	
	Native Soils		→ *Baseflow/Interflow*
		↓ *Groundwater*	

Figure 2.1: Schematic of Bioretention Cell Physical Components

The upper bowl varies in depth, which is usually between 150mm to 300mm (Hunt et al, 2012). Upper bowls are not deeper than that for a variety of reasons: ensuring vegetation health, safety concerns, and compaction issues (Hunt et al, 2012). Usually, there is a distinct inflow point into the upper bowl which may be a point source, such as a pipe, or sheet flow via a broad crested weir. There should also be means to safely channel flow from the BRC in a downstream direction, usually by a broad crested weir, in the case of overflow, when inflow exceeds the infiltration capability and upper bowl storage volume of the BRC.

Besides the upper bowl, volume storage is also available in the filter media below it. The actual available volume depends on volume of the BRC and the porosity of the media. When the BRC is saturated, there is no available storage. Field capacity of the filter media is the porosity of the media at the point when all the water has been able to freely drain out of the soil. This is the porosity that should be considered crucial for design purposes. Total dryness of the soil is not possible due to capillary forces which will hold some water between soil particles (Freeze and Cherry, 1979). The soil at the bottom of a BRC would not be exposed to solar energy and other potential drying conditions that a soil at the surface would be exposed to. Media bowls are typically 0.6m to 1.2m deep (Davis et al, 2009). BRCs with the largest areas and volumes have the least number of overflows. This is likely due to the ability to have volume available (all other factors being equal) for subsequent events due to greater infiltration and evapotranspiration between events (Hunt et al, 2012).

Most BRCs consist of gravel, soil mixture of sand and vegetative soil, mulch and plants (Gulbaz and Kazezyilmaz-Alhan, 2016). Recommended filter media mixes vary greatly. Some have a large organic content, such as in Delaware, USA, which recommends equal thirds of sand, peat moss and mulch (Davis et al, 2009). However, specifications, such as those in North Carolina, USA, call for only 3% to 5% organic materials with the remainder mostly sand (Davis et al, 2009). In other cases, volumetric sand content is approximately 88% to allow filtration but not clogging and 8% is topsoil with 4% leaf litter or mulch (Willard et al, 2017). Mulch is usually added on top to contain moisture for the plants (Gulbaz and Kazezyilmaz-Alhan, 2016). Some studies found that media with high organic content can have a negative result for treatment of nutrients, such as phosphorus (Hatt et al, 2009). Leaching of phosphorus from the media can lead a higher concentration of phosphorus in the outflow compared to the inflow. The addition of amorphous iron and aluminum, to be discussed in Section 2.3.4.2, has shown evidence in decreasing the concentration of nutrients. It should be noted that where possible, local soils available in the vicinity of the planned BRC should be used. There is not much sense in requiring unnecessary importation of material over a long distance (Gulbaz and Kazezyilmaz-Alhan, 2016).

2.3.3.2 Underdrains

BRCs can be constructed with or without an underdrain. Underdrains are typically 10cm diameter perforated plastic pipes wrapped in filter cloths to prevent clogging by fine soil. The pipe and filter cloth combination are laid in a bed of clear stone gravel. (DeBusk, 2011). Figure 2.2 illustrates the many potential configurations for underdrains. If the native soils are permeable soils, such as sand and gravel, there may be no need for an underdrain (see Case A, Figure 2.2), provided there is a safe means to convey overflow from the BRC. If the soils are more cohesive, such as silt and clay, then an underdrain is recommended so that water can flow from the BRC when the inflow rate exceeds the lower infiltration capability of the soil underlying the BRC or if there is a high groundwater table (see Case

B, Figure 2.2). It is important to realize that the BRC must be able to empty itself in reasonable time to have sufficient empty volume available to receive runoff from the next precipitation event. The underdrain can be located at the bottom of the BRC, as shown in Case B of Figure 2.2, or at some vertical mid-point, as shown in Case C of Figure 2.2.

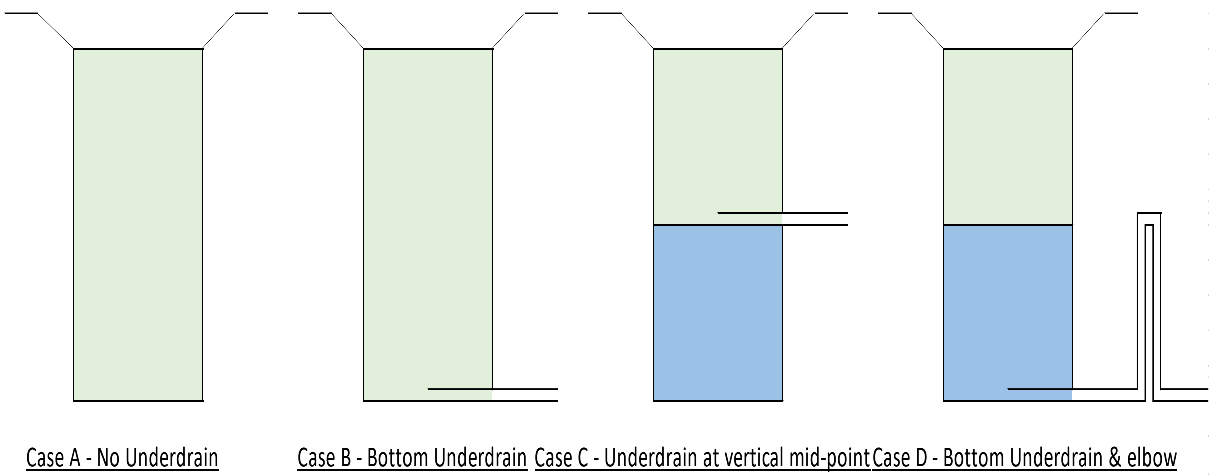

Figure 2.2: Underdrain Configurations within a Bioretention Cell

If the underdrain is located at the bottom of the BRC, the full depth of the BRC can be realized in the flowpath where quality treatment will occur; whereas, if the underdrain is located at some vertical mid-point, then the flow path will be reduced. However, an Internal Water Storage (IWS) will be created underneath the invert of the underdrain. The depth of an IWS can vary from 0.3 m (DeBusk, 2011) to 0.97m (Geheniau, 2015). Water captured in the IWS will never return to surface flow, and therefore, will be forced to infiltrate into the soil below and adjacent to the BRC. This aspect can have a large performance impact as the water below the elevated underdrain and pollutants contained therein will never have an impact on the receiving water bodies.

Another version may have an underdrain at the bottom of the BRC; however, downstream piping may have an elbow/valve assembly that would effectively create an IWS, yet still allow the full flowpath of water quality treatment in the BRC for the water that does not get infiltrated into the ground, as shown in Case D of Figure 2.2 (Geheniau, 2015). In a study of three different BRCs, this type of BRC performed the best where only 14% of the inflow left the BRC as surface flow, underdrain flow included. The formulization of volumetric dynamics of BRCs has been attempted by using the concept of Bioretention Abstraction Volume (BAV) (Davis et al, 2012). BAV could be used as a design parameter in the design of BRCs.

It is interesting to note that in some studies, underdrains were installed in order to sample outflow (Hatt et al, 2009). However, the very presence of the underdrains affects the water quantity and water quality performance of the BRC.

2.3.3.3 Design Aspects

For long established infrastructure, such as storm sewers, approval agencies have design criteria in place which must be adhered to in order to obtain approval for construction. Since BRCs are relatively new infrastructure, no such design criteria are present. However, there are some guides available for a practicing engineer in Southern Ontario to reference, such as the *LID Planning and Design Guide* (TRCA & CVC, 2010). Various design aspects need to be considered, as follows:

- What size rainfall event should be targeted?
- How can treatment efficiencies be evaluated?
- Optimization for treatment of specific pollutants or nutrients, such as phosphorus
- BRC surface area to drainage area ratio
- BRC location
- BRC volume to runoff volume ratio for targeted rainfall event
- How fast can the BRC drain to have available storage volume for the next rainfall event
- Infiltration rate of media
- Porosity of infiltration media

One of the key design aspects is targeted precipitation event. This can range from 13 mm (Willard et al, 2017) (DeBusk, 2011) to 25 mm (LSRCA, 2016). In the case of a BRC at Villanova University, USA, where 25 mm was the targeted event, it was found that

80% of the annual watershed rainfall input was removed from surface waters and similar pollutant percentage removal was observed (Davis et al, 2009).

In 1994, the Ministry of Environment Conservation and Parks (MECP), then known as just the Ministry of Environment (MOE), produced the *Stormwater Management Practices Planning and Design Manual* (MOE, 2003). It was updated in 2003 as the *Stormwater Management Planning and Design* (SWMPD) *Manual* which is still in use today and is one of the most referred to document in SWM in Ontario. The SWMPD manual promotes the use of treatment trains of SWM Best Management Practices (BMPs) and discussed BMPs at the lot level, conveyance level and the end-of–pipe level. However, over the years it has been largely used as a reference for the design of SWM ponds.

It should be noted that many of LID techniques listed in Section 1.2 are, in fact, mentioned in the SWMPD manual, including soakaway pits and infiltration trenches, which have similar characteristics as BRCs if they are constructed in a permeable soil and provide ground infiltration. The SWMPD manual has, amongst others, the following design criteria for soakaway pits and infiltration trenches, which are also relevant for BRCs:

- They can be used in soils with a percolation rate ≥ 15 mm/hr. (percolation rate is determined by a standard field test, typically done for septic field design where 15 mm/hr is typical for sandy loam and less cohesive soils will have higher percolation rates. (MOE, 2003))
- The minimum volume is to contain 15 mm of rainfall event with a four-hour duration
- Drawdown time (the time from the end of a precipitation event to elimination of ponding) of 24 to 48 hours
- Infiltration trenches can be utilized for drainage areas less than 2 ha.

Note that some BRC guidelines call for BRCs to drain within 72 to 96 hours for the design rainfall (Davis et al, 2009).

BRCs are typically designed for drainage areas less than 0.8 ha (Davis et al, 2009). A BRC could service a small drainage area, as long as there would be sufficient water to maintain whichever type of vegetation is planted in it. In a controlled outdoor study of

BRCs, each BRC had a surface area representing 2.9 % of the drainage area (Gulbaz and Kazezyilmaz-Alhan, 2016). This is greater than the minimum of 2.5% of the drainage area of the Virginia Stormwater Management Handbook but less than the suggested design ratio of 5% to 7% (DeBusk, 2011). The BRC surface area to drainage ratio is important as observed in a study in Australia (Hatt et al, 2009). In that study, one of the BRCs had a surface area of only 1% of the impervious tributary area and exhibited overflow in 11 of 28 observed storm events and therefore limited its effectiveness.

From a common-sense standpoint, it would seem logical to have a BRC located at the downstream end of a drainage area, to take advantage of natural gravitational surface flow. Similarly, a BRC should also be located such that there is a suitable overflow path for safety reasons and also a suitable underground outlet if there is an underdrain in the BRC. If BRCs are constructed on sloped ground, pooled water would not be able to collect in the upper bowl; therefore, it would make sense that BRCs should be sited on near level ground.

Whether or not a BRC is located on private land or on land of a public agency is an issue. Maintenance of a BRC can come into question if located on private land. Agreements or easements can be drawn to encumber the owner to maintain the BRC, but enforcement can also be costly. Therefore, a risk analysis may be required. Consider the theoretical scenario where, for example, ten BRCs are proposed in a LID project and there is no guarantee of maintenance but 90% of the facilities are anticipated to be maintained. Should the entire program be rejected, and associated opportunities lost just because one out of ten may not be properly maintained?

In some studies, it was found that media with slow infiltration rates created greater peak flow reduction and greater increase in lag time, which subsequently improved water quality treatment. However, if the rate is too slow, the BRC will fill up with water too fast for the available room and overflow will occur. Therefore, there are trade-offs between hydraulic performance and pollutant removal. It is particularly important to target the purpose of the BRC and then design the BRC appropriately to that targeted goal within site constraints (Hatt et al, 2009). In other words, if a BRC is designed for water quantity management, as opposed to water quality management, design parameters may differ.

2.3.4. Bioretention Cell Performance

2.3.4.1 Water Quantity

The negative effect of urbanization on the hydrologic response of the land during precipitation events is evident when comparing the pre-development and post-development states, and the associated repercussions on the environment. Figure 2.3 illustrates typical pre-development and post-development hydrographs. The LID principle strives to return the hydrologic response back to the pre-development state. There are three water quantity control goals to consider when attempting to carry out that principle, as follows:

- Decrease of peak flow
- Decrease of runoff volume
- Increase in lag time between the inflow of runoff into the BRC and the outflow of runoff from the BRC

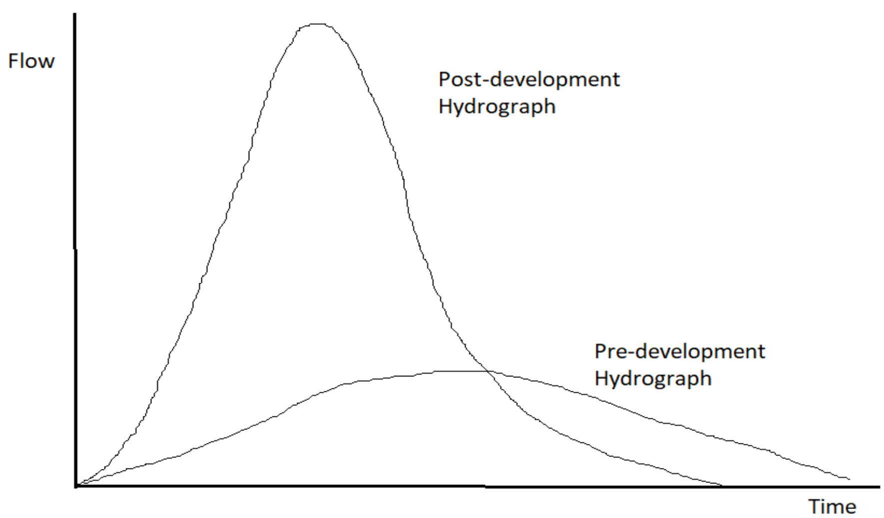

Figure 2.3: Typical Pre-development and Post-development Hydrographs

Municipal infrastructure, such as curb gutters, catch basins, storm sewers, concrete channels lead to faster flow velocity. This in turn creates greater peak flow in the post-development state. Increased imperviousness also increases the runoff volume, as there is less opportunity for water to infiltrate into the ground, amongst other abstractions. Faster flow velocity of municipal infrastructure also reduces the time of concentration of flow and flow from different paths congregates much quicker, and coinciding peaks can have a compounding effect.

The ability of a BRC to reduce peak flow can be dependent on the volume of the BRC and the media permeability (Khan et al, 2012(part 1 and 2)). Volume of the BRC can be divided into the surface pooling volume and the volume of the media. The greater the volume, the greater the reduction of peak flow due to a hydraulics principle known as routing. The volume of the filter media can be further divided into active volume and IWS. Water captured in the IWS will never reach the outlet so this would naturally help reduce peak flow exiting the BRC. In an Australian study in which none of the BRCs had IWS an 80% reduction of peak flow was observed (Hatt et al, 2009). In a controlled outdoor study of four BRCs, it was found that greater organic content reduced the peak flow more than a higher sand content (Gulbaz and Kazezyilmaz-Alhan, 2016).

In a study in 2017 of 23 rainfall events of a mature BRC it was observed that runoff volume reduction was between 37% and 100% (Willard et al, 2017). The 100% runoff volume reduction occurs in low volume rainfall events as all the runoff volume is absorbed by porosity in the media. In a study of six BRCs in North Carolina and Maryland, 20% to 50% volume was lost due to infiltration and evapotranspiration (Li, 2009). Volume reduction can also be solely evapotranspiration, especially in hot climates. For example, in a study in Australia it was estimated that, on average, 33% of runoff volume to a BRC was lost due to evapotranspiration. However, other studies indicate a more modest rate, for example, 10% in mid-southern USA (Hunt et al, 2012). The greatest challenge when treating big storm events is the size limitation. LID facilities are not large enough to handle the large volume of runoff from infrequent and high intensity storm events (Davis et al, 2012). For that purpose, BRCs maybe best be paired with other detention facilities, such as dry detention ponds, that create an overflow treatment train configuration to handle the extra volume that overflows or bypasses the BRC (MOE, 2003).

Typically, the time of concentration for rainfall runoff for a parking lot can be in the range of five to ten minutes; however, with flow through a BRC, that time can be increased by hours due to the lag time from when water enters a BRC and exits it, even if no volume is lost. The lag time can range from 60 minutes to 600 minutes (Kratky, 2017). This has a beneficial hydrological effect in minimizing congregation of flow and coinciding peaks which can have a compounding effect to increase flow and potentially cause floods in downstream locations. In the controlled outdoor study of BRCs, it was discovered that there is a longer lag time of peak flow for BRCs with greater organic content than those with greater sand content (Gulbaz and Kazezyilmaz-Alhan, 2016). The decreased permeability of organic soils slowed down the flow.

Other hydraulics observations of BRCs included the observation that the decrease of peak flow and increase of lag time became less pronounced as the runoff intensity increased (Gulbaz and Kazezyilmaz-Alhan, 2016). In that same study, it was discovered that a deeper ponding depth created less peak flow treatment and a lower increase of lag time, likely due to the increase in hydrostatic pressure within the BRC (Gulbaz and Kazezyilmaz-Alhan, 2016). Another study found similar results in that hydraulic performance of BRCs tend to decrease with increased rainfall depths and duration (Li, 2009). This is likely due to the limited capture volume of BRCs. BRC’s performance is highest for small precipitation events and annual water balance analyses, since most rainfall events have less volume than the targeted volume of between 13mm or 25 mm rainfall events. For example, in the Greater Toronto Area, the average precipitation event is 5 mm (Behera et al, 1999).

2.3.4.2 Water Quality

Besides water quantity, water quality is another important topic for study in the performance of BRCs. The pollutant topics most studied are as follows:

- Total Suspended Solids (TSS)
- Heavy metals, such as zinc, lead and copper
- Nutrients, mainly nitrogen and phosphorus

Other extraneous influences that can sometimes be a factor in water quality are oils & grease, bacteria, chlorides, and temperature; however, they were not consistently addressed in the reviewed studies.

Section 2.1 lists all the processes that can occur in a BRC to treat water quality, from detention to heat transfer. Volume retention is quite significant as it also takes water quality into consideration. The outflow pollutant loading will, in most cases, be automatically reduced if the volume of water exiting the BRC to the receiving body of water is significantly reduced through infiltration into the ground surrounding the BRC. Therefore, a distinction should be made between lowering of a concentration of a pollutant or total mass reduction. For example, consider the following general concept of an inert pollutant. If a BRC only acts as a filter, then the concentration would be affected. In some events, a high concentration of pollutant may be present in the inflow and some of it captured reducing the concentration since a consistent volume of stormwater would continue to pass through; however, in a subsequent event with a lower pollutant concentration in the inflow some of the pollutant previously held in the BRC may be released thereby temporarily increasing the concentration. Overall, the BRC would have a buffeting effect on the water body's reception of the pollutant. By contrast, if a BRC is constructed in a non-cohesive soil, or the BRC has an IWS, then there would be a certain portion of the pollutant in the stormwater that would never reach a receiving water body due to infiltration and thereby the total mass of the pollutant is reduced.

Detention, which increases lag time, is also an important aspect. Most of the other process listed in Section 2.1 will be more effective the longer the pollutant laden stormwater is filtered through the BRC. However, if the stormwater is held too long in the BRC, then there is the risk that the BRC will not be sufficiently empty to receive runoff from the next precipitation event and may result in overflow.

BRCs are highly effective in capturing TSS. It is the gradation of the media, which is the most important aspect, while the vegetation has only a modest benefit (Hunt et al, 2012). TSS is caught in the top 10 to 20 cm (Willard et al, 2017) (Khan et al, 2012a). Removal of TSS is varied in studies, ranging from 47% to 100% (Khan, 2012b).

As with TSS, heavy metals are caught in the upper 10 cm to 20 cm of the media (Hunt et al, 2012). This top layer of the media should be removed and replaced as part of a regular maintenance program. The heaviest concentration of metals is at the flow inlets into the BRCs (Hunt et al, 2012). If the media has a high pH level, metal absorption tends to be higher (Hunt et al, 2012).

Regarding nutrients, nitrogen and phosphorus, it is important to realize that nutrients themselves, in small concentrations, may not be pollutants as they are naturally occurring in the environment. The problem is that if the concentration and total mass loading of nutrients is too high for the environment to naturally process, they become considered as pollutants. Results of treating nutrients varies greatly due to complexity of several factors including type of filter media, depth of media, temperature, volume capture and concentration (Khan, 2012b).

With respect to nitrogen, biological nitrification-denitrification is the primary process in nitrogen removal. Most denitrification occurs in the upper layer of the media, above any IWS (Willard et al, 2017). Denitrification requires a reasonable residence time to occur (DeBusk, 2011). Since nitrogen is a nutrient, a large amount of it can be removed by vegetation uptake. Cold temperatures reduce the rate of nitrification-denitrification process (Hunt et al, 2012). Studies have shown occurrences of nitrogen leaching out of the media so there is an actual increase in nitrogen flowing out of the BRC than there is entering into it (Khan, 2012b). Therefore, selection of the proper media is very important.

Phosphorus can be treated in three ways: filtration of particle bound phosphorus, chemical sorption, and by vegetation uptake of the nutrient. Field studies have shown varied results from -7% to 99% removal (Willard et al, 2017). As discussed previously, care must be taken to ensure there is no phosphorus available in the media itself, as it will leach out, resulting in a negative outcome in phosphorus removal. Amorphous iron and aluminum have proven to be successful additives in the media in regards to phosphorus sorption (Hunt et al, 2012). Both of these substances are by-products from municipal water treatment facilities (Hunt et al, 2012) and therefore may be conveniently available for this purpose. Effective phosphorus removal is time dependent in unsaturated conditions and if an IWS is present in the BRC, its top water level is recommended to be at least 45cm to 60 cm below the top of the BRC (Hunt et al, 2012). Long term studies have shown that mature plants have the ability to reduce phosphorus by uptake of nutrients in growth (Willard et al, 2017).

2.3.5. Bioretention Cells in Cold Climate

Results of studies of BRCs in cold climates has been varied. The main issue is that soil moisture near the surface may freeze thereby potentially reducing infiltration rates. A further concern regarding winter performance is the added loading of sediment due to application of sand and salt on the roads for traffic safety reasons. A study of BRCs in cold climate conditions in Calgary demonstrated that there was not a substantial drop in hydraulic performance (Khan, 2012). In that study, stormwater did not disperse evenly as in unfrozen conditions, but instead, still found a path of least resistance through the frozen material. It was also found that there was not a significant change in water quality treatment performance.

In a study of a BRC in cold climate in Montreal (Geheniau et al, 2015), the results were different than those in the aforementioned study in Calgary (Khan et al, 2012). During warm weather, the BRC had a volume retention efficiency of 59.7%; whereas, during cold weather, the volume retention efficiency dropped to 35.0%. It should be noted that different methods were used in the Calgary and Montreal studies that could influence the outcome. In the Calgary study stormwater was inputted into the BRC from a tank to simulate a rainfall event; whereas, the Montreal study used actual runoff as an input. As a result, the Montreal study would include snowmelt events and also antecedent moisture conditions resulting from inter-event timing would be a factor. In the Calgary case there was time between the experiments (the events) for the opportunity of the moisture content in the BRC to return to field capacity.

The cold climate studies show that BRCs still have the capability to treat stormwater quantity and water quality in cold weather conditions but the issue of the level of drop of between warm climate and cold climate conditions needs further study, especially over long periods of time (Kratky, 2017).

2.3.6 Construction, Operation and Maintenance

2.3.6.1 Construction

To promote infiltration into native soil below a BRC, scarifying the native soil with the teeth of a backhoe bucket is recommended (Hunt et al, 2012). BRCs should be constructed late in the construction process after surrounding land has been sodded and paved. If BRCs are constructed too early, the sensitive growing media may be compacted by construction equipment and thus, reducing their designed capabilities. There is also the potential that BRCs may be subjected to sediment, eroded from the construction site and unintentionally deposited in the BRC, thereby compromising its performance. If the BRC shell is excavated at an earlier stage, its inlet should be isolated and protected from receiving any flow during construction.

2.3.6.2 Operation and Maintenance

As previously mentioned, many studies show that TSS and metals accumulate in the top 10 cm to 20 cm of the BRC`s filter media (Willard et al, 2017) (Khan et al, 2012a). Therefore, this upper layer should be frequently replaced to avoid clogging. The USA Environmental Protection Agency gives detailed guidance on replacing BRC media in their operation and maintenance guide (USAEPA, 2016). Long term studies have shown BRCs to perform at a high level since any natural compaction of the filter media is offset by roots and activity of fauna, such as worms (STEP, 2019).

BRCs are known to make an aesthetic contribution to the environment through landscape design. If there is this expectation from a BRC, regular maintenance is required such as, pruning, debris removal, weeding and mulching.

2.3.7 Summary

LID, with its principle of trying to mimic natural hydrologic functions of the earth, has established itself in the field of water resources. Of the many techniques in the LID `toolbox`, BRCs is one of the most popular. This literature review has presented and summarized many of the studies conducted on BRCs.

Some common benefits and trends have surfaced in the review of BRCs. One of those is the ability of BRCs to hydraulically handle a majority of minor precipitation events. In some cases, all of the runoff from a minor event can be absorbed by the BRC. Large variations in the design of BRCs were observed with corresponding performance variations. However, there is limited evidence of BRC performance on large, infrequent major storm events, such as a 100-year storm. As a result, the best scenario is a treatment train consisting of LID facilities to handle most events with supporting complementing infrastructure such as underground chambers or dry detention ponds to handle large precipitation events. There seems to be a lack of studies to attempt to consolidate these SWM principles for the full and complete SWM treatment scenario.

Another common trait is the ability for BRCs to treat water pollutants. In most cases, the longer a polluted water is filtered through the BRC; the better the treatment of many of the pollutants. However, there is a trade-off in that if the water is retained in the BRC too long then there will be insufficient empty volume available to accept the next precipitation event. More research and modelling would benefit the fine tuning of these conflicting principles.

If a model could be created that would fine tune the design of BRCs and also incorporate a full SWM treatment train package to deal with all hydrologic and hydraulic aspects, then we would be a step closer to creating ideal SWM solutions that benefit and sustain the environment.

2.3.8 Knowledge Gaps

Since LID techniques are relatively new, they have been subjected to many studies to better understand them. For example, BRCs have been studied under laboratory conditions (Li, 2008) (Gulbaz and Kazezyilmaz-Alhan, 2017) and in field applications with controlled water input (Khan et al, 2012). Real precipitation events have in some cases been used in the monitoring of BRCs (Davies et al, 2012) (Willard et al, 2017).

The research that was reviewed only studied isolated BRC facilities. However, there appears to be a lack of evidence of the performance of BRCs and other LID techniques with overall stormwater management objectives of a low-density residential development site.

There are many design criteria for stormwater management of a residential development that SWM ponds traditionally satisfied according to agency design criteria. Can the use of LID and accompanying treatment trains satisfy agency design criteria and eliminate the need for a SWM pond? Zimmer et al (Zimmer, 2007) attempted to address this question in 2007 however had various limitations, especially addressing infiltration. More is now known and new advanced modelling techniques present more options to represent performance of LID facilities than in 2007 and such an updated exercise is warranted.

From the research literature, there is no doubt that there is environmental benefit to utilizing LID techniques, such as BRCs, for stormwater management, especially regarding water quality. However, can LID techniques along with other treatment train measures be used to eliminate a SWM pond, or at least, reduced the size of the SWM pond to have more efficient land use in the development? Can the implementation of BRCs fulfill the objectives of a water balance analysis, such as mitigate the loss of groundwater infiltration due to urbanization? The objective of this study is to attempt to answer these questions by analyzing the pilot project using computer software that has single event and continuous modelling, such as PCSWMM, supplemented with on-site monitoring and observations (which were used to calibrate and validate the numerical simulations). Also, what are the initial cost and operational implications of stormwater management with BRCs and reduced SWM detention storage versus a traditional SWM retention storage?

Chapter 3: Methodology

3.1 Site Description and Location

Prior to development of the Mosaik Glenway subdivision, the site consisted of farmland, several medium sized farm structures, a residence with an attached garage and a vacant dwelling (Soil Engineers, 2011a). The proposed development drastically changed the drainage characteristics by increasing imperviousness resulting in the increase of stormwater runoff flow, runoff volume and various water quality impacts. The development included construction of typical urban infrastructures, such as roads, sanitary sewers, watermains, storm sewers and normally, stormwater management ponds.

Mosaik Glenway development is an 11.7 ha residential land subdivision consisting of 154 lots creating 123 single detached family homes and 62 semi-detached homes, along with a community park (Schaeffers, 2014 and Dillon, 2012). The legal land description of the site is part of Lot 95, Concession 1 West of Yonge Street, of the Town of Newmarket (Dillon, 2012). The subdivision was developed by Mosaik Glenway Homes Inc. and managed by Paul Bailey, Bazil Developments Inc. The site has a unique location since it is located at the west boundary of the Town of Newmarket, immediately southeast of the intersection of Davis Drive West and Bathurst Street, as shown in Figure 3.1, which is from an aerial photograph of the site during development. The site has moderately rolling topography, generally ranging from 0.5% to 2.2 %, with the lowest part of the site containing a watercourse at the northwesterly corner of the property, immediately beside the above-mentioned intersection. Besides draining most of the property, this watercourse, which is a tributary of the West Holland River, drains an area of 39.6 ha, west of Bathurst Street and conveys flow northerly via a culvert under Davis Drive.

Figure 3.1: Site Location

3.2 Geotechnical and Hydro-Geotechnical Aspects

A thorough geotechnical investigation was conducted in 2011 (Soil Engineers, 2011). The site is located in the Schomberg Clay Plain, at the border with the Oak Ridges Moraine (Chapman and Putnam, 1984). The Schomberg Clay Plan is a partially eroded drift, filled with sand, silt and lacustrine clay as a result of glacial movements (Soil Engineers, 2011, Dillon 2012)). Field work consisted of drilling 11 boreholes in July 2011 using a track-mounted, continuous flight powered auger. In seven of the boreholes, the soil discovered under the topsoil was a silty clay and in the remaining boreholes, a silty clay till was discovered (Soil Engineers, 2011). Of the boreholes that were drilled, boreholes 3,4,6,7 and 8 are the ones most related to the BRCs of this research. The depth of the groundwater in those 6 boreholes are summarized in Table 3.1 (Dillon, 2012). Note that the geotechnical report indicated that some of the groundwater elevations are the result of a perched groundwater table.

Table 3.1: Groundwater Levels in Key Boreholes

Borehole Number	Borehole Depth (m)	Soil Colour Change Brown to Grey	Seepage Encountered During Augering		Measured Groundwater/Cave-in Level*on Completion	
		Depth (m)	Depth(m)	Remarks	Depth (m)	Elev. (m)
3	6.6	4.5	6.0	Some	1.5/3.0*	275.5/274.0*
4	6.2	6.2+			3.0 / 3.7*	273.5 /272.8*
6	7.8	7.0			6.0*	274.5*
7	10.8	5.5			6.0	276.5
8	12.3	9.0			6.4	276.8

Note: * denotes Cave-in level (In wet sand and silt layers, the level generally represents the groundwater regime at the borehole location)

In addition to a geotechnical investigation, a hydro-geological analysis of the site was also conducted. In addition to the 11 boreholes done by Soil Engineers, Dillon Consulting Limited drilled an additional 11 monitoring wells at 8 locations, which were monitored from March to September 2012. Some of the sites had two monitoring wells at different depths. (Dillon, 2012). Monitoring results from the wells that had two depths indicated an upward gradient at certain times of the year indicating a groundwater discharge area. Monitoring Wells 3 and 4 (shallow and deep wells, respectively) are the wells important for the present research.

Constant head tests were done at these locations to help determine the saturated hydraulic conductivity (K) values at the screened locations. At Monitoring Well 4 (deep) the value of K was low such that a steady drawdown could not be established, thus a low-flow pumping transient-rising test was used. In some cases, K had to be estimated using the Hazen correlation (Freeze and Cherry, 1979). The measured and calculated values of K indicate a soil that will not allow water to pass through it easily. (Dillon, 2012)

In addition to measured and calculated analyses for K, in-situ infiltration tests were also conducted. Infiltration tests 2,4,6, 7 and 8 are significant to this study. The results of those infiltration tests for the test holes of interest are listed in Table 3.2 (Dillon, 2012). A value of K is required to represent the entire site, and this was determined to be 7.3 x 10^{-8} m/s by arithmetic mean of past studies.

Table 3.2: Results of Infiltration Tests

Infiltration Test Location	Infiltration Rate (mm/hr)	Saturated Hydraulic Conductivity (K) (m/s)
INF2	8.5	1.1 x 10-7
INF4	1.2	4.3 x 10-8
INF6	6.1	3.8 x 10-7
INF7	8.9	3.6 x 10-7
INF8	59.2	4.1 x 10-6

3.3 Stormwater Management

The low area near the intersection of Bathurst Street and Davis Drive discussed in Section 3.1 would be a prime location for a traditional SWM pond because of the low elevation. However, for urban planning reasons, a SWM pond at this location was not desirable but instead, a park was preferred at this location. The challenge was to design a SWM scheme that would meet the necessary requirements (such as peak flow control, water quality treatment, erosion control, water balance, etc.) without utilizing a traditional SWM pond of this location.

The design of the SWM system was initiated by Schaeffer & Associates Ltd. (Schaeffers), a local, medium sized private consulting engineering firm, in 2011. Municipal agency approval was received in 2014. The municipal infrastructure, such as watermains, sanitary sewers, storm sewers, SWM facilities and roads, was constructed primarily by the main contractor, Con-Drain Company (1983) Ltd. Construction of the municipal infrastructure was completed in 2015. Subsequent house construction was finished in 2017.

The SWM scheme consists of an array of LID techniques, including exfiltration of perforated storm sewers pipes, bioretention swales and rain gardens. In some cases, these LID techniques are complimented with other more traditional SWM facilities, such as oil/grit separators, to establish treatment trains. Final water quantity control for large precipitation events is handled by a large DoubleTrap™ underground storage tank under the park (Schaeffers, 2014). Figure 3.2 identifies the various subareas of the subdivision and how they are treated by SWM and Figure 3.3 is a photograph of the storage tank under construction.

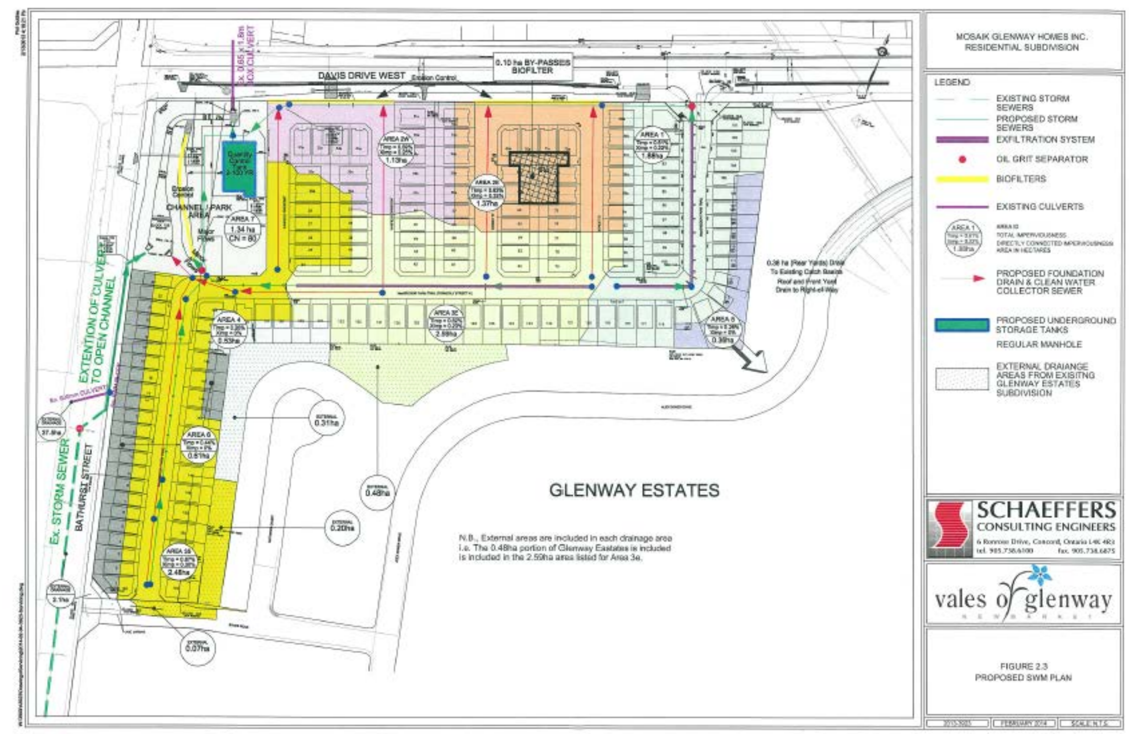

Figure 3.2: SWM Plan (Courtesy: Schaeffer & Associates Ltd.)

Figure 3.3: Photograph of Storage Tank under Construction (Courtesy: Schaeffer & Associates Ltd.)

3.4 Bioretention Cells

The design includes three BRCs each of which is unique. One is a large cell in the low area of the subdivision, near the northwesterly section and is referred to as the Bathurst BRC since it parallels Bathurst Street, although there is a wetland between Bathurst Street and the BRC. The other two BRCs are considerably smaller and narrower. These other two BRCs are located along the northerly limit of the subdivision, adjacent to Davis Drive, and are referred to as Davis East BRC and Davis West BRC. Davis East BRC has a relatively flat profile whereas the Davis West BRC is on a more sloped topography and required terracing. The above three bioretention facilities are the subject features of this research.

Besides the physical ways the BRCs are constructed, which will be discussed in the next two sections, they also differ in regard to their sizes and the areas they service. The Bathurst BRC is approximately 700 m^2 in area whereas the Davis BRCs are both about 280 m^2, therefore, the Bathurst BRC is more than twice the area of each of the Davis BRCs. The sub-catchment areas flowing into the BRC is also varied as can be seen in Table 3.3.

Table 3.3: Comparison of Sub-Catchment Areas to Areas of BRCs

	BRC Area	Sub-Catchment Area	Ratio of Sub-Catchment Area /BRC Area	Percentage of BRC/Sub-Catchment
	(m^2)	(ha)	Ratio	(%)
Bathurst BRC	700	5.11	73:1	1.37
Davis West BRC	280	1.13	40:1	2.48
Davis East BRC	280	1.37	49:1	2.04

As presented in Section 2.3.3.3, BRCs are typically designed for drainage areas less than 0.8 ha (Davis et al, 2009). The percentage of BRC surface area to drainage area for the Bathurst BRC is far less than the minimum of 2.5% of the Virginia Stormwater Management Handbook (DeBusk, 2011). The BRC surface area to drainage ratio is important as observed in a study in Australia (Hatt et al, 2009). In that study, one of the BRCs had a surface area of only 1% of the impervious tributary area and exhibited overflow in 11 of 28 observed storm events. As can be seen in Table 3.3 the Bathurst BRC is servicing an excessively large area and has a low percentage surface area to sub-catchment area.

However, BRCs are often categorized by their directly connected impervious areas to BRC surface area ratios (STEP, 2019). The directly connected impervious areas would be roads, sidewalks and driveways. This analysis would exclude pervious areas which provide infiltration, initial abstraction and surface storage. It would also exclude impervious roof areas since roof downspouts outlet to the ground and eventually on to pervious areas. Therefore, water draining from roofs has an opportunity to infiltrate into the ground after it flows on to the pervious area and thereby not being a directly connected impervious area. This type of directly connected impervious area analysis may be more indicative of performance than comparing to the entire sub-catchment area. To account for this, the updated values are listed in Table 3.4.

Table 3.4: Comparison of Directly Connected Impervious Areas to Areas of BRCs

	BRC Area	Directly Connected Impervious Area	Ratio of Directly Connected Impervious Area: BRC Area	Percentage of BRC Area: Directly Conn. Impervious Area
	(m^2)	(m^2)	Ratio	(%)
Bathurst BRC	700	11400	16:1	6.1
Davis West BRC	280	2500	9:1	11.6
Davis East BRC	280	3000	11:1	9.3

A third comparison could be made between the storage volumes. Storage volume can be categorized into four components: upper surface bowl storage, filter media storage, stone storage above IWS and IWS itself. A porosity of 0.4 is assumed for volumes of storage in filter media and stone whereas, 100% of the volume of the surface bowl can be used. Table 3.5 summarizes the total storage of each BRC and calculates the ratio of directly connected impervious area to BRC storage volume.

Table 3.5: Comparison of Directly Connected Impervious Areas to Volumes of BRCs

	Directly Connected Impervious Area	BRC Total Storage Volume	Ratio of Directly Connected Impervious Area: BRC Volume
	(m^2)	(m^3)	Ratio
Bathurst BRC	11400	378	30:1
Davis West BRC	2500	190	13:1
Davis East BRC	3000	302	10:1

Again, the Bathurst BRC statistics is the least favourable and the Davis BRCs are closely aligned. This is not only due to the large drainage area it is servicing but also it is restricted in depth due to the seasonally high groundwater table and hence had volume restrictions. The underdrain in the Bathurst BRC is offset from the bottom of the BRC by only 0.1m whereas, the west and east Davis BRCs have underdrain offset distances of 0.35 and 0.45m, respectively. Therefore, based on the values of Table 3.5, the Bathurst BRC will likely perform to a lower level, as compared to the two Davis BRCs. The challenge from a volume perspective is more pronounced than the challenge from just an area perspective, when comparing Tables 3.3, 3.4 and 3.5.

3.4.1 Bathurst Bioretention Cell

The Bathurst BRC drains two distinct drainage areas, one from the south, known as 3S, and from the east, known as 3E, as shown in Figure 3.2 and 4.2. Flows to the Bathurst BRC are collected by street catch basins and conveyed to storm sewers. The Bathurst BRC is part of a treatment train whereby initial water quality treatment is provided by an oil/grit separator, in this case, a Stormceptor^{TM}. The purpose of the oil/grit separator is to remove heavy suspended solids to reduce the load of sediments the BRC may receive and help prevent prematurely clogging. Suspended solids captured by the oil/grit separator can be removed by a vacuum truck on a regularly scheduled maintenance program.

Water from the oil/grit separator then drains into a control chamber/manhole that has two outlet pipes. One outlet pipe is 450mm Ø that outlets to the BRC. The second pipe is a 1200 mm Ø with an invert that is 450 mm higher than the first pipe. The reason for this elevation difference is to limit flows to the BRC. Once flows get greater than the free flow capacity of the 450mm Ø pipe the flows by-pass the BRC and are directed to the large storage tank. This way the BRC will not be inundated with major system flows.

Flows enter the BRC at its southerly end by pipe flow from the 450mm Ø pipe. The flow of water continues longitudinally through the BRC. Flows can leave the BRC by one of the following routes:

1. Filtration through the filter media, past the underdrains and infiltrate into the native surrounding soils
2. If the above filtered water flows are greater than the infiltration capability of the mineral sub-soils, then the underdrains will convey flows to a free outlet
3. Evapotranspiration
4. If there is too much water, then flows over the berm at the north end of the BRC will occur

Figure 3.4 is a photograph of the Bathurst BRC and Figure 3.5 is a cross-section of it.

Figure 3.4: Photograph of Bathurst BRC, on the left, and Wetland, on the right

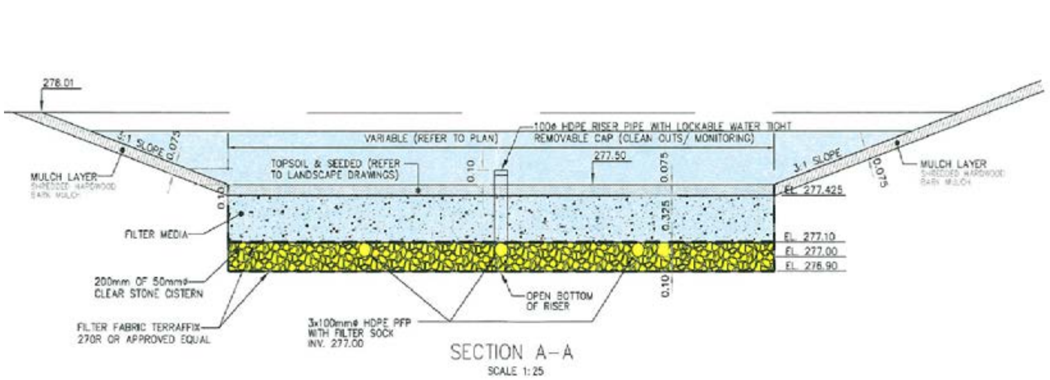

Figure 3.5: Cross-Section of Bathurst BRC (Courtesy: Schaeffer & Associates Ltd.)

3.4.2 Davis East and West Bioretention Cells

These two BRCs are similar but unique from the Bathurst BRC. Firstly, they receive flows from surface drainage; not piped flow. The streets of the crescents that are tributary to these BRCs have no catch basins. The northerly sections of the crescents have a single cross-fall, so they do not collect water at the inside corners compared to a situation where those sections of roads have crowned cross-sections. Water enters the BRCs via curb cuts and then vegetated strips, along the northerly curb of the northerly sections of the crescent. The purpose of the vegetated strips at the curb cuts are to capture some of the suspended solids prior to entering the BRCs. Ditch Inlet Catch Basins are installed at the outlets to safely convey large flows from a large storm that exceeds the treatment capacity of the BRC. BRCs are not designed to handle such large storms alone and require back-up support from other infrastructure. Figure 3.6 illustrates a schematic of a typical Davis Drive BRC.

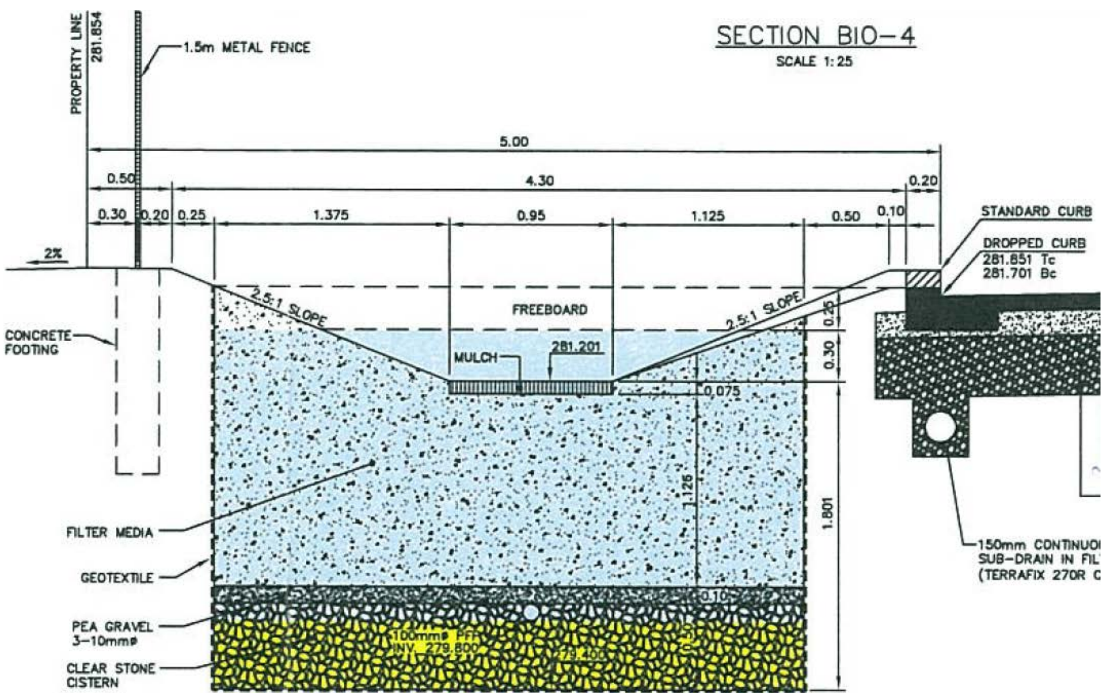

Figure 3.6: Typical Cross-Section of a Davis BRC (Courtesy: Schaeffer & Associates Ltd.)

Figure 3.7: Photograph of a Davis BRC showing Curb Cuts

3.5 Bioretention Cell Monitoring

In November 2017, a proposal for monitoring the site was made by STEP. The purpose of the monitoring program is to evaluate the performance of the SWM system implemented at the Mosaik Glenway site (STEP, 2016). While this broader monitoring program has no direct relationship with this thesis, data from this monitoring program was used in this thesis to calibrate and validate the computer simulations described in Section 3.6.

Along with the monitoring by STEP, Ryerson University also planned to do some monitoring of the stormwater infrastructure at the site. However, the monitoring by Ryerson University is specifically in regard to the storm sewer exfiltration/groundwater infiltration component of the system. Therefore, there are a number of unique aspects to the Mosaik Glenway SWM system that are of interest to researchers and conservation authorities.

A field monitoring program was initiated on a trial basis in 2017 and fully implemented in 2018. For the BRCs, orifice standpipes, incorporated with the underdrains, were installed during construction at upstream and downstream ends of the BRCs (STEP, 2016). Water level loggers, such as *Solinst 3001 Levelogger ®5*, were placed in those standpipes to record water levels every 5 minutes. Besides the level loggers, water samplers, temperature loggers and conductivity loggers were also to be installed (STEP, 2016).

To complement the above monitoring equipment, equipment to record precipitation was also utilized. On-site rain gauges were used in conjunction with rain gauges located at the office of LSRCA, which is located approximately four kilometers from the site.

3.6 Computer Modelling

3.6.1 Evaluation Criteria

To evaluate stormwater management with BRCs for the site, typical design criteria for stormwater quantity control required by the local conservation authority was considered. In reviewing the LSRCA's *Technical Guidelines for Stormwater Management Submissions,* three primary stormwater quantity control criteria were recognized, as follows: firstly, maintain water balance such that infiltration of pre-development conditions is maintained in post-development conditions; secondly, erosion control is exercised such that the volume of a 25 mm rainfall event is released over a 24 hour period; and thirdly, peak flows from design storms of 2, 5,10, 25, 50 and 100 year storms, with four durations, under post-development conditions do not exceed peak flows of respective storms under pre-development conditions. A computer model needs to be chosen that will accommodate all the above criteria. To accomplish the above, computer simulation models are typically created for pre-development conditions, post-development conditions without mitigation, and finally, post-development conditions with mitigation by SWM facilities which may include LID techniques.

3.6.2 Choice of Model

Various models were considered for this thesis. The model that was chosen is PCSWMM. It is a commercial version of the Storm Water Management Model (SWMM) created by the US Environmental Protection Agency. SWMM is a hydrological/hydraulics computer model commonly used by researchers and professional engineers in the field of water resources engineering (Lee et al, 2017). SWMM was created in 1971and has had many revisions and updates over the years, including the addition of modeling of LIDs in 2010 (James, 2010). It is basically a model that computes runoff from single precipitation events or continuous rainfall patterns for both, quantity, and quality perspectives. PCSWMM has added on extra user features to SWMM, primarily through graphic interface enhancements and it is marketed by Computational Hydraulics International, who provided a student license for this research.

In 2018, a detailed review was done of computer software models that model LIDs. (Kaykhosravi et al, 2018). In the review, eleven computer models were evaluated for their ability to model hydrologic and hydraulic processes associated with LIDs. In that review, PCSWMM was noted for having the following features which are relevant to this study:

- Ability to generate peak flow hydrographs
- It can model a site at the detailed design level including infrastructures such as storage facilities, orifices and weirs
- Variable time steps can be used
- Ability for detailed explicit modeling of LIDs
- LIDs can be defined within a sub-catchment or can be defined as an individual sub-catchment
- Variability to model evaporation by different methods
- Variability to model infiltration by different methods, although it excludes Richards equation
- Can model runoff from a sub-catchment as run-on to a LID

However, the review did have reservations in that some simplifications were made of hydrologic and hydraulic process in LIDs. For example, Richards equation is more detailed in calculating unsaturated flow in soil and it is not used in PCSWMM. Also, the review noted the fact that PCSWMM cannot model detailed vegetation interaction with rainfall and is noted as a potential weakness (Kaykhosravi et al, 2018). PCSWMM can only model two soil layers of a LID but that is not a hindrance for this research.

Another model that was attempted for this study is the Treatment Train Tool (TTT) developed by STEP. This model also uses SWMM as its base “engine”. TTT is primary a planning tool with its major strength being in water balance. However, it fails to provide some of the detailed information that a model like PCSWMM can provide, such as, peak flow hydrographs, infiltration flows etc. TTT would potentially have more application in the design field if it had these additional capabilities.

In this application, PCSWMM uses principles of conservation of mass, energy and momentum (Rossman, 2015) to balance the physical processes of precipitation, infiltration, evapotranspiration and runoff. When precipitation lands on a sub-catchment’s surface, the simulation of some of the above physical processes begins and only after stored surface water exceeds depression storage is there the potential for runoff (Rossman, 2015). This process is repeated over time increments until an outflow hydrograph of runoff versus time can be created. More detail will be provided in Section 4.2

3.6.3 Rainfall

First, continuous rainfall was modelled in this study. In this case, all recorded rainfall from April 1, 2018 to November 30, 2018 was modelled, by PCSWMM on a continuous basis. Output from this model was compared to the monitoring data. Results from the continuous modelling will provide input into the water balance analysis.

One of the most important inputs into a hydrological model, like PCSWMM, is precipitation, in this case, rainfall. For guidance, the technical guidelines of the local conservation authority, LSRCA, was referenced (LSRCA, 2016). In that document, the Chicago, four-hour duration, design storm and the Soil Conservation Service (SCS), Type II, twelve-hour duration, design storm must be modelled for return periods of 2, 5, 10, 25, 50 and 100 years.

These requirements make sense since the Chicago, four-hour duration, design storm distribution reflects a high peak intensity of a short duration intense summer storm, whereas the less intense, but higher volume, SCS, Type II, twelve-hour duration, design storm is more akin to a frontal weather system. Both types of rainfall situations should be considered when studying BRCs due to the different ways they can challenge a SWM system and also because they represent the two major types of precipitation events seen in the Greater Toronto Area. If a storm is of high intensity over a short duration, as in a typical summer storm, absorptive responses of soil of the BRC may be a factor. Conversely, the SCS storm of more drawn-out time period may allow absorptive characteristics of the soil of the BRC to be more effective; however, the higher volume of the SCS storms may be more critical in the design volume of a SWM facility that has to control the system outflow such that it is not greater than pre-development flow.

Snowfall is not considered in this study since it is not usually a consideration in the design of a SWM system of a subdivision from a quantity control aspect. Also, monitoring at the site in 2017 and 2018 was discontinued during winter months.

Besides large storm events, some smaller event storms will also be considered, specifically 25 mm, 12.5 mm and 5 mm events. For example, LSRCA requires that a 25 mm rainfall event with a Chicago, four-hour duration, design storm distribution be considered for erosion control in the design of a SWM facility. For erosion control, the volume from this storm must be detained and released over a minimum 24-hour period. A 25 mm rainfall event is also a criterion for volume control in regard to quality treatment, along with 12.5 mm and 5 mm events. A 5

mm event can be considered an average rainfall event in the Greater Toronto Area (Behera et al, 1999). LSRCA criteria requires that subdivision infrastructure be designed such that no flow leaves a site as runoff from a 5 mm rainfall event (LSRCA, 2016).

3.6.4 Evapotranspiration

Another input into PCSWMM is evapotranspiration. Evapotranspiration is not important for modelling of single design storm events due to the relatively short duration of time involved in the storm, but it can be a significant component in modelling continuous periods of weather, particularly during warmer months when evapotranspiration is highest, as in this case, from April 1 to November 30. Up to two thirds of precipitation can result in evapotranspiration in some circumstances in a water balance analysis (Delidjakova et al, 2014). It is hard to measure evapotranspiration, as compared to stream runoff which is a more physically evident hydrologic process. Regarding dealing with evapotranspiration in the BRCs, it was deemed that using just evaporation should suffice since the plants in the BRCs had just been planted and therefore immature.

There are five methods of dealing with evaporation in PCSWMM: constant rate, time series, directly from a climate file, computed from temperatures and monthly averages (James et al, 2010). For this study, the monthly rates method was chosen since relevant information was available from a detailed study done at a site in the Greater Toronto Area, approximately 25 km. away from the subject site. This detailed study (Delidjakova et al, 2014) was conducted in 2014 and it reported evapotranspiration at three different locations of three different land uses. The site known as Kortright was physically the most similar to the study area of this research. Using these monthly rates was presumed to be most relevant of the five possible methods for modelling evapotranspiration, as specified in PCSWWM. However, the rates in this study were observed and not pan evaporation rates as required in PCSWMM (James et al, 2010) Therefore, adjustments had to be made to the monthly values of the above detailed study of the Kortright site. To supplement those values, pan evaporation rates were used from a study in the United States (Farnsworth and Thompson, 1982). The results of a monitoring site that had the closest latitude and longitude to Toronto was used. A comparison was made with the pan evaporation rates of the U.S. study with evapotranspiration rates of the detailed study of the Kortright site. A relatively consistent ratio was derived in that comparative analysis and as a result, the observed monthly evaporation rates of the detailed study of the Kortright site were upscaled by a factor of 1.47 to represent pan evaporation rates. These final monthly rates were inputted for the months of April 1 to November 30 in the climatology editor of PCSWMM and are listed in Table 3.6.

Table 3.6: Pan Evaporation Rates inputted into PCSWMM

Month	**Pan Evaporation Rate (mm/d)**
April	1.780
May	3.623
June	4.778
July	5.595
August	4.680
September	3.100
October	1.953
November	1.137

3.6.5 Surface Infiltration

As in the case of evapotranspiration, surface infiltration plays an important role in continuous modelling. To model infiltration, there are several options that can be used in PCSWMM. Some of the methods of modelling infiltration, such as the Horton Method, were tried. The Horton method created flows that would not normally be expected for this type of case when trial design storms were run, based on past results of similar situations, and it was ruled out. This decision was supported by design literature (ASCE, 1992). After testing some of the options, the Green-Ampt method gave results more in line with what would normally be expected, based on results of similar situations. The Green-Ampt method requires three input parameters, namely, initial soil moisture deficit, K and suction head. As was discussed in Section 3.2, a K value of 7.3 x 10^{-8} m/s was determined to represent the entire site. This value was inputted into the model. For values of the other Green-Ampt parameters, values for silty clay and silty clay till were taken from user guides of PCSWMM (James et al, 2010) The values between the two soil types were weighted according to the number of boreholes of each soil type.

3.7 Cost Examination

In regard to LID techniques, there are uncertainties about associated capital and long-term maintenance costs (STEP, 2013). The costs of the infrastructure assessed in this research will be evaluated from capital cost perspectives, and maintenance and operation costs to help resolve some the uncertainties. Unit costs will come from a variety of sources. The most reliable source will come from the actual tender of the contractor who constructed the BRCs at the Mosaik Glenway site. To supplement the tender costs, the manufacturers of various infrastructure items, such as storage tanks and oil/grit separators will be contacted. For operation and maintenance costs of BRCs, documents from conservation authorities will be consulted (STEP, 2013).

In regard to stormwater volume storage, a notable distinction should be made. In most suburban residential developments, stormwater volume storage is usually done by SWM ponds with permanent pools. In these ponds the permanent pool is to provide water quality treatment. Active storage is available on top of that permanent pool to temporarily detain volumes resulting from storm flows being restricted to pre-development peak values.

In this study the typical SWM pond will be substituted by oil/grit separators and an underground storage tank. The oil/ grit separators provide water quality control, and the storage tank represents the available active storage that would exist above a permanent pool in a SWM pond. This substitution is being done for several reasons. One of the reasons is due to economics.

The value of land is extremely variable and very dependent on location. For example, the value of serviced land in the urban core of the City of Toronto is far greater that the value of land in the satellite community of the Town of Newmarket, which is still considered a part of the Greater Toronto Area, but is located approximately 45 km north of downtown Toronto. Using a concrete storage tank, as was used in the Mosaik Glenway subdivision, eliminates this highly variable land cost aspect. Land on top of the tank was used as a park so all land cost regarding the SWM facility can be effectively ignored in this analysis. There were also some technical engineering reasons for using a storage tank instead of SWM pond and these will be discussed in Section 4.2.3.3.

A concrete storage tank will be assumed for this study since it was accepted by approval authorities for the Mosaik Glenway site. A case could be made to use other types of products, such as plastic chambers, but that would unnecessarily complicate the analysis. The status quo of the Mosaik Glenway site is considered acceptable. Figure 3.3 is a photograph of the storage tank being constructed at the Mosaik Glenway site.

For an oil/grit separator, a product called a *Jellyfish*, by Imbrium, was chosen for the costing exercise. Again, a case could be made to use another product; however, the *Jellyfish* was chosen since it is known to be acceptable to approval authorities to provide sufficient water quality treatment to satisfy local requirements (MECP, 2016). By way of an example, a copy of an Environmental Compliance Approval (ECA) (Number 3221-AC8LZK) from MECP was obtained by public online records (MECP, 2016). In that case, two *Jellyfish* units were utilized. One of the *Jellyfish* units, JF-6-5-1, was used to service an area of 2.07 ha, and therefore, would be an appropriate model for one of the sub-catchments draining to the Davis BRCs.

Chapter 4: Results and Discussion

4.1 Monitoring

4.1.1 Site Reconnaissance

Various site visits were made during this study, particularly in the spring of 2019. First, on May 9, 2019, the site was visited prior to an anticipated rainfall event. Also, on May 10, 2019, a site visit was made immediately after the rainfall event of 13.6 mm occurred. On May 12, 2019, two days after the rainfall event, the site was again visited.

During that site reconnaissance, it was observed that the Bathurst BRC was holding water and had developed wetland features, for example, emergent vegetation, such as cattails, had been established. The development of wetland features (resulting in a permanent or near-permanent pool of water in the BRC bowl) could be the result of a combination of the following: excessive drainage area in comparison of the area of the BRC; relatively small IWS; sediment on top of mulch (post construction cleaning had still not been done at that time) and/or potential groundwater contribution. During those site visits, the water level in the Bathurst BRC increased approximately 15 cm but came down 10 cm after approximately two days post rainfall event. Conversely, the Davis East and West BRCs did not display any standing surface water immediately after the rainfall event. Therefore, all the stormwater that drained from the sub-catchments to the Davis East and West BRCs had infiltrated down into the BRCs.

It should be noted that although the Bathurst BRC was not performing as intended as a BRC, stormwater would still be receiving water quality treatment. For example, water exiting the facility through the underdrain would still have flowed through the filter media and thereby undergone a filtration process. Also, the fact that the facility has taken on wetland characteristics would be a benefit since there would be an uptake of nutrients as wetland vegetation grows.

4.1.2 STEP Monitoring

Some complications were encountered during monitoring. During a meeting on June 20, 2019 with various parties involved with monitoring at the site, it was reported that initially there had been grading issues at the curb cuts; the sod had been installed too high. As a result, water was not flowing into the BRCs in the intended fashion. Normal operation of the BRCs were not realized until this grading issue was quickly resolved on the site and the sodding at the curb cuts was regraded.

Monitoring data was received and reviewed. In the data, there are various gaps and inconsistencies which resulted from equipment issues. The data that is of best use is the data for Davis East BRC. Figure 3.7 is an excerpt of the data from monitoring equipment at the Davis East BRC from July 1, 2018 to August 30, 2018. The rainfall hyetograph is shown in the upper part of Figure 4.1, and the depths of water in the upper bowl surface storage and the bottom of the BRC are shown in the bottom part of Figure 4.1. It should be noted that there is a gap in the data of the wells in late July and early August, likely due to equipment malfunction.

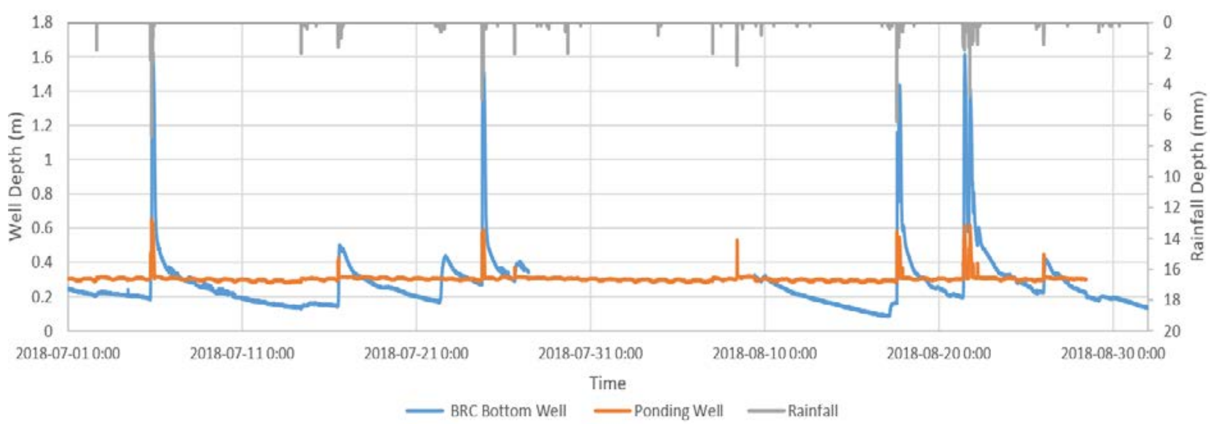

Figure 4.1: Monitored Water Levels in the Davis East BRC, July and Aug. 2018

4.1.3 Summary and Discussion of Monitoring

Initially, the infiltration rate for the bottoms of the BRCs were from calculations of the infiltration rate based on information from the geotechnical reports. As a result of those calculations, the infiltration rates for Bathurst BRC, the Davis West BRC and Davis East

BRC were 4.9 mm/hr, 19.6 mm/hr and 8.0 mm/hr, respectively. However, the results of the STEP monitoring indicated that these values were not indicative of the true infiltration characteristics of the soil, as will be next discussed.

One documented method of calculating infiltration rate for an observed situation is to record the time from when an enclosed storage facility drains from 75% full to 25% full (75%/25%) (CivilWeb Spreadsheets, 2021). This method eliminates the extreme head situations when the enclosed space has the maximum head and when the head in the enclosed space is dwindling down to zero. As can be seen by the plot of water surface elevations in the Davis East BRC in Figure 4.1, water levels and corresponding infiltration flows decrease exponentially, as the head approaches zero. Eliminating this dwindling phenomenon is appropriate if a single numerical rate of infiltration is required, as a parameter in PCSWMM.

Calculating the infiltration rate from the above-described method would result in the following formula:

Equation 1: Calculation of Infiltration Rate at Bottom of BRC

$$f = \frac{V_{p75\text{-}p25}}{A_{p50} \times t_{p75\text{-}p25}}$$

However, since the horizontal area of the BRCs are consistent, the volume, V, of the numerator of this formula can be altered to Area x distance (A x d), as follows:

$$f = \frac{A \times d_{p75\text{-}p25}}{A \times t_{p75\text{-}p25}}$$

The Area, A, term can then be cancelled out and just difference of depths in the BRC over a period can be used to estimate the infiltration rate, f, as follows:

$$f = \frac{d_{p75\text{-}p25}}{t_{p75\text{-}p25}}$$

Unfortunately, the 75%/25% split scenario does not present itself since a subsequent rainfall event will add more stormwater to the storage facility before the volume can fall to the 25% level. From the observed monitoring data of the Davis East BRC, the time from July 6, 2018, 22:25 to July 14, 2018 17:25, the water level decreased by 0.197 m during that time period. Calculations regarding this event resulted in an infiltration rate of 1.05

mm/hr. A similar event scenario developed between July 17, 2018 8:45 to July 22, 2018 resulting in a decrease of water level in storage of 0.16 m for a comparable infiltration rate of 1.4 mm/hr. An average of 1.27 mm/hr of these two events will replace the calculated infiltration rate for Davis East BRC that was used from the geotechnical reports in the design process. Therefore, the infiltration rate is reduced from 8 mm/hr to 1.27 mm/hr or by a factor of 0.16. Since there is no similar data for the other BRCs, the factor of 0.16 is the best available information and the infiltration rates for the other BRCs were reduced by a factor of 0.16 from the calculated values to better represent actual conditions, rather than the calculated infiltration parameters, when it comes to computer modelling.

4.2 Computer Modelling

4.2.1 Parameters

The study area was divided into four primary sub-catchment areas. Figure 4.2 illustrates the study area as shown in the PCSWMM model.

Figure 4.2: Satellite view of site, as shown in PCSWMM, illustrating the four primary sub-catchment areas

The four primary sub-catchment areas are shown in light green shading in Figure 4.2. However, the primary sub-catchments were then subdivided into components representing the fact that roof downspouts were disconnected from the conveyance system and roof water drained onto pervious surfaces for potential infiltration. Impervious surfaces of roads and driveways were considered directly connected to the conveyance system of gutters, catch basins and storm sewers, whereas lawns and roof areas were modelled as indirectly connected to the conveyance system. Most of the physical parameters for computer modelling, such as sub-catchment areas were taken from the engineering drawings by Schaeffers. The parameters used for modelling are summarized in Table 4.1.

Table 4.1: PCSWMM Sub-Catchment Parameters

	3S		**3E**		**2W**		**2E**	
Parameter	**DCI**	**IC**	**DCI**	**IC**	**DCI**	**IC**	**DCI**	**IC**
Area (ha) Internal	0.41	1.46	0.73	2.59	0.25	0.88	0.3	1.07
Length (m)	300	300	350	350	140	140	130	130
Width (m) (calc'd)	14	49	21	74	18	63	23	82
Slope (%)	0.37	0.37	1.20	1.20	2.14	2.14	2.08	2.08
Imperviousness (%)	100	58	100	51	100	47	100	53
n Impervious	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013
n Pervious	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
D_{store} - Impervious.	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
D_{store} -Pervious.	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
Subarea Routing	OUTLET	PERVIOUS	OUTLET	PERVIOUS	OUTLET	PERVIOUS	OUTLET	PERVIOUS

Note:
DCI denotes directly connected impervious
IC denotes indirectly connected

4.2.2 Continuous Precipitation Simulation for Water Balance Analysis

Included in the objectives of stormwater management is preserving groundwater and baseflow of watercourses (MOE, 2003). Maintaining baseflow in watercourses is necessary for aquatic life (LSRCA, 2016). In order to do this, an attempt is made to replicate the hydrological cycle. The hydrological cycle has input of precipitation and output of evapotranspiration, infiltration and surface runoff, as shown in Figure 4.3.

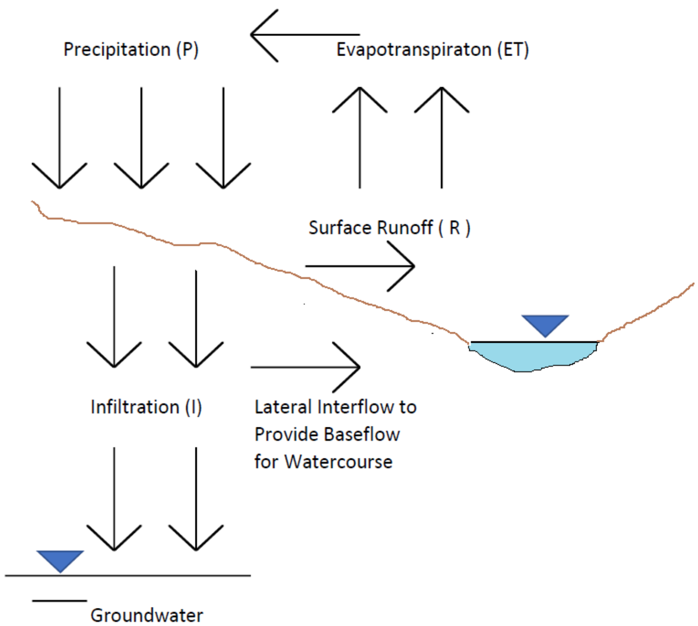

Figure 4.3: The Hydrological Cycle

The hydrological cycle is represented by a simple water balance equation:

Equation 2: Water Balance

$$P=ET+I+R$$

Where P=precipitation (mm)

ET=evapotranspiration (mm)

I=infiltration (mm)

R=runoff (mm)

Often an intermediary step is taken where evapotranspiration is subtracted from precipitation to determine net surplus (also known as effective or excess precipitation) and then net surplus is divided into infiltration and surface runoff. The most traditional way of doing this analysis is using a method developed by Thornthwaite and Mather, 1957 (MOE, 2003). Various tables have been produced to provide typical annual values for components of the hydrological cycle, for example, in the SWMPD Manual (MOE, 2003). In that table, annual evapotranspiration, infiltration, and surface runoff are given depending on the type of soil and land use.

For this research, only the growing season between April 1 and November 30 was considered, as was done by STEP monitoring. Continuous precipitation is available for the period of April 1 to November 30, 2018. The resulting amount of infiltration was derived from PCSWMM modelling.

PCSWMM was initially run for the pre-development condition to determine the amount of infiltration that occurs between April 1and November 30, 2018. Then PCSWMM was run for the post-development conditions which provided values such that the amount of infiltration loss can be calculated due to the increase of imperviousness resulting from the construction of roads, driveways, sidewalks, landscape paving and dwellings of urban development. Finally, BRCs were added to the post-development model to determine how much mitigation of the loss of infiltration is provided by the BRCs to attempt to fulfill the objective of matching pre-development infiltration.

4.2.2.1 Pre-Development

In most water balance analyses the entire site is analyzed as a single unit. In Section 3.4 it was discussed that the Bathurst BRC was designed under less-than-ideal circumstances in that it had a large sub-catchment area draining into it and the percentage of BRC area to sub-catchment area was the least favourable of the BRCs. As a result, the study of the contribution of BRCs to water balance was done using two approaches: study of the entire site and study of only the Davis BRCs. In order to facilitate this approach, the four sub-catchments of the post-development configuration were maintained. Table 4.2 provides the results of that modelling.

Table 4.2: Results of Pre-development Water Balance

SUB-CATCHMENT IDENTIFICATIONS	3S	3E	2W	2E	TOTAL
Area (m^2)	18700	33200	11300	13700	76900
INPUTS (per Unit Area)					
Precipitation (mm)	655	655	655	655	655
OUTPUTS (per Unit Area)					
Evapotranspiration (mm)	110	102	98	93	102
Net Surplus (mm)	545	553	557	562	553
Site Infiltration (mm)	386	378	357	362	374
Runoff (mm)	159	175	201	199	179
INPUTS (Volumes)					
Precipitation (m^3)	12249	21746	7402	8974	50370
OUTPUTS (Volumes)					
Evapotranspiration (m^3)	2063	3379	1103	1278	7825
Net Surplus (m^3)	10186	18367	6298	7695	42545
Site Infiltration (m^3)	7212	12561	4030	4963	28766
Runoff (m^3)	2974	5806	2268	2732	13779

4.2.2.2 Post-Development Continuous Model, without Bioretention Cells

Urban development increases imperviousness by the construction of impermeable roofs of dwellings, paved landscaped areas, private driveways, sidewalks and paved roadways. This extra imperviousness reduces the amount of infiltration into the ground which ordinarily could be conveyed through the surface to the groundwater table, laterally to provide baseflow to a watercourse, or in some cases, even be conveyed back to the surface for evapotranspiration through capillary forces. This next step of modelling attempts to model these physical responses to determine the loss of infiltration on the site due to urbanization.

In the next model, the sub-catchments were further discretized, as discussed in Section 4.2.1 into sub areas of directly connected impervious areas and indirectly connected areas. There is no infiltration from the directly connected impervious areas. The amount for water available for infiltration on the pervious zones of the indirectly connected areas is the precipitation onto the pervious areas plus also the runoff from the impervious areas connected to them, such as rooftops, since the downspouts are disconnected from the conveyance system. This arrangement is possible through the subarea routing option when the sub-catchment parameters are established. Table 4.3 summarizes the results from the post-development modelling without BRCs.

Table 4.3: Results of Post-development Water Balance without Bioretention Cells

SUB-CATCHMENT IDENTIFICATIONS	3S		3E		2W		2E		TOTAL
	DCI	IC	DCI	IC	DCI	IC	DCI	IC	
Area (m^2)	4100	14600	7300	25900	2500	8800	3000	10700	76900
INPUTS (per Unit Area)									
Precipitation (mm)	655	655	655	655	655	655	655	655	655
OUTPUTS (per Unit Area)									
Evapotran. (ET) (mm)	185	146	181	138	173	129	172	133	147
Net Surplus (m^3)	470	509	474	517	482	526	483	522	508
Site Infiltration (mm)	0	156	0	178	0	184	0	162	133
Runoff (mm)	470	354	474	339	482	342	483	359	375
INPUTS (Volumes)									
Precipitation (m^3)	2686	9563	4782	16965	1638	5764	1965	7009	50370
OUTPUTS (Volumes)									
Evapotran. (ET) (mm)	759	2132	1324	3582	432	1137	517	1425	11312
Net Surplus (m^3)	1927	7431	3457	13383	1206	4627	1448	5583	39058
Site Infiltration (m^3)	0	2270	0	4600	0	1621	0	1738	10228
Runoff (m^3)	1927	5161	3457	8783	1206	3006	1448	3846	28830

As can be seen when comparing Tables 4.2 and 4.3, the amount of water infiltrated into the ground was reduced from 28,766 m^3 to 10,228 m^3. Therefore, there is a shortfall of 18, 538 m^3 to be made up by other measures to meet the objective of no loss of infiltration due to urban development.

4.2.2.3 Post-Development Continuous Model, with Bioretention Cells

The next step was to find out how much the BRCs were able to mitigate the loss of infiltration due to urbanization. The BRCs were then added to the previously described model. It should be noted that there are two ways of modelling BRCs in PCSWMM. They can be incorporated as a component of a sub-catchment or be considered 100% of an independent sub-catchment. For this study, after considering both methods, the latter scenario was chosen. The second scenario was chosen because it allowed clearer tracking of flows from tributary areas to the BRCs.

Prior to discussing the water balance results, it is worth revisiting the discussion on infiltration at the bottom of the BRCs, as discussed in section 4.1.3. Figure 4.4 is a plot from PCSWMM of hyetograph, infiltration and runoff, for Davis East BRC using the design infiltration rate. In contrast, Figure 4.5 is a plot of hyetograph, infiltration and runoff, using the modified infiltration rate that was calculated from monitoring results.

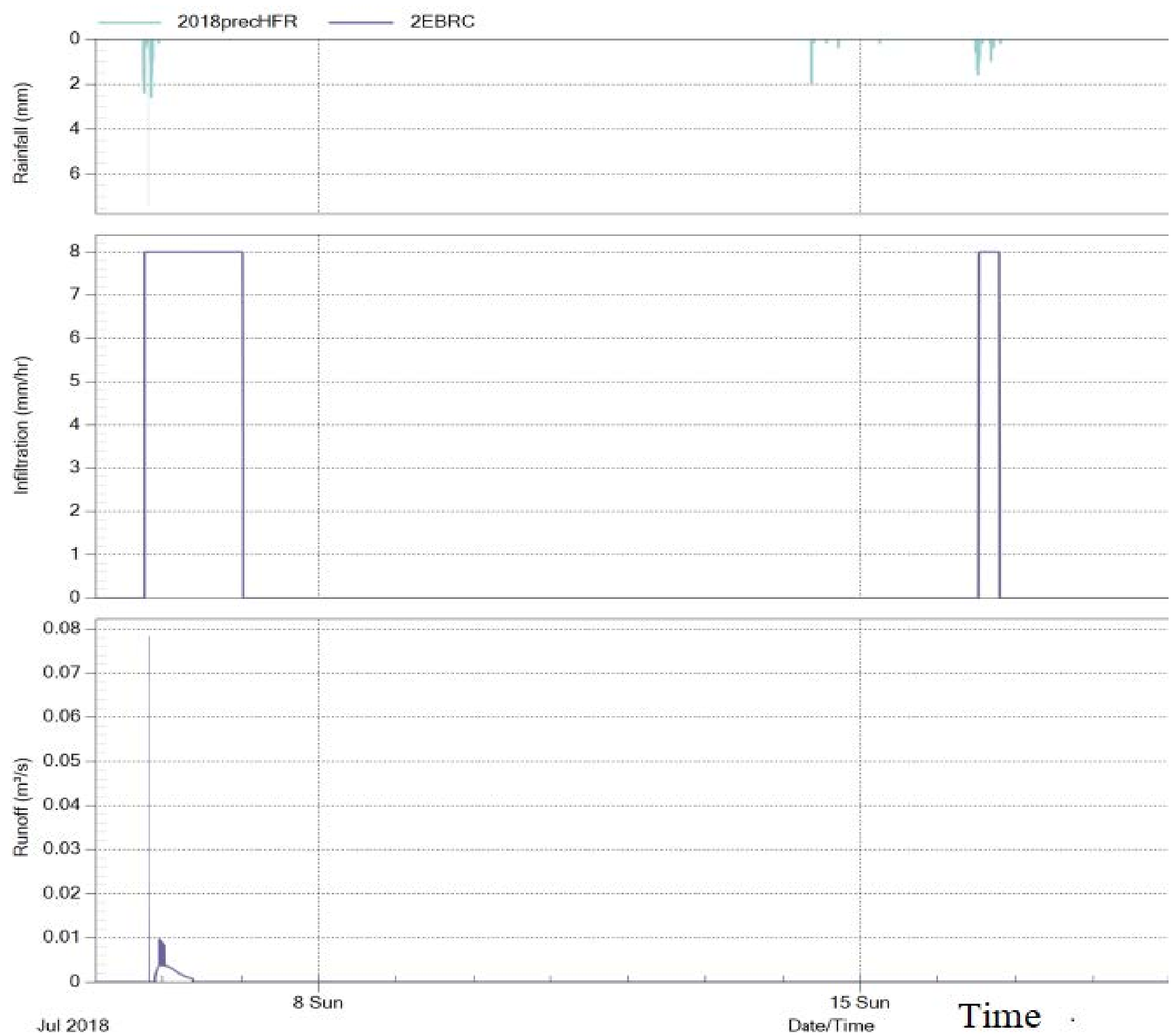

Figure 4.4: PCSWMM Plot of Hyetograph, Infiltration and Runoff for Davis East BRC using the Design Infiltration Rate

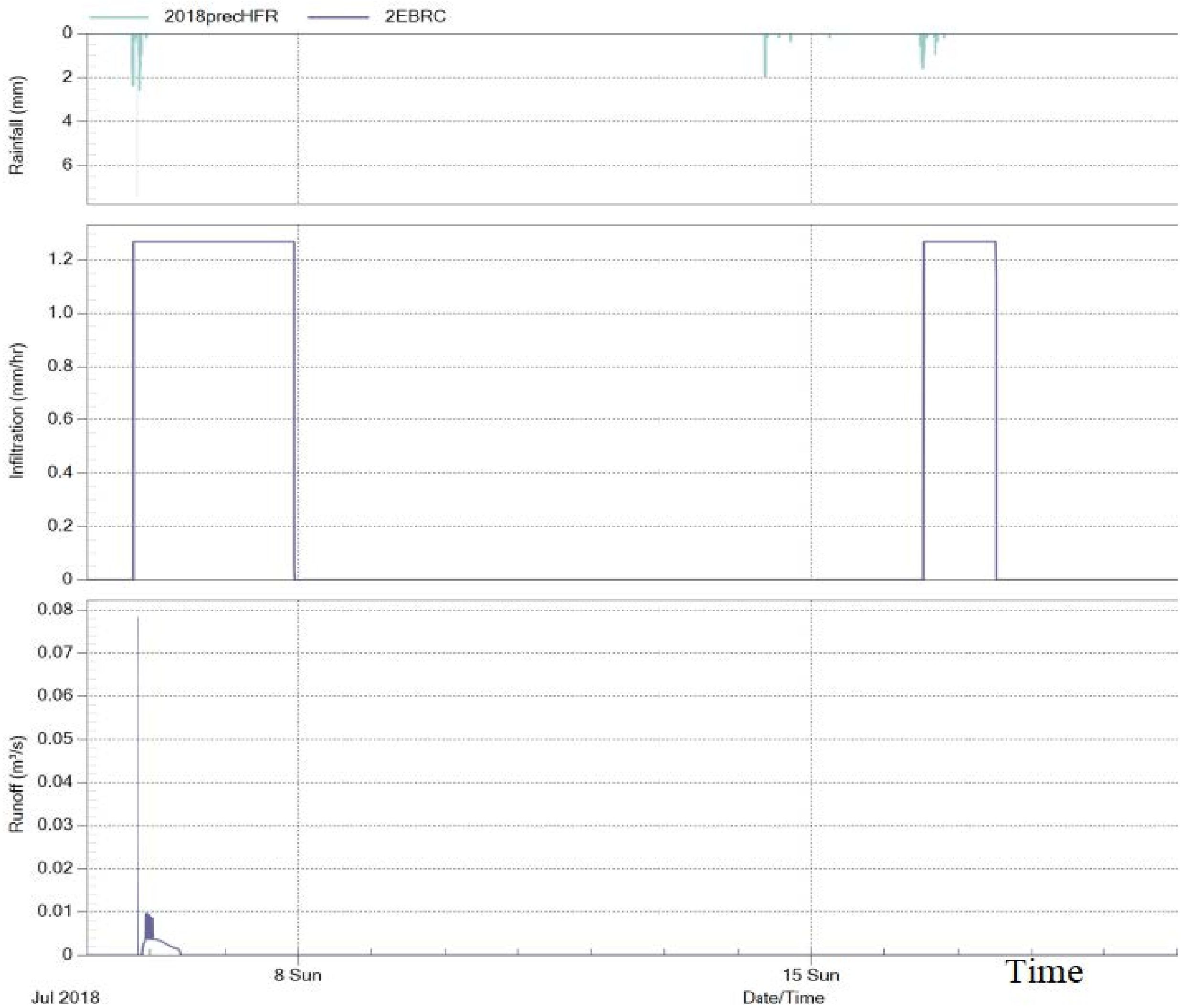

Figure 4.5: PCSWMM Plot of Hyetograph, Infiltration and Runoff for Davis East BRC using the Modified Infiltration Rate from Monitoring

When comparing Figures 4.4 and 4.5, it can be seen that the length of time for the BRC to empty after the rainfall event lasted longer with the modified lower infiltration rate, however, PCSWMM did not model the infiltration phenomena as it actually occurred, as shown from the monitoring in Figure 4.1 in Section 4.1.2.

Table 4.4: Post-Development Continuous Model with BRCs

CATCHMENT IDENTIFICATIONS	3S		3E		2W		2E		Bioretention Cells			
	DCI	**IC**	**DCI**	**IC**	**DCI**	**IC**	**DCI**	**IC**	**3BRC**	**2WBRC**	**2EBRC**	**TOTAL**
Area (m^2)	4100	13900	7300	25900	2500	8500	3000	10400	700	300	300	76900
INPUTS (per Unit Area)												
Precipitation (mm)	655	655	655	655	655	655	655	655	655	655	655	655
OUTPUTS (per Unit Area)												
Evapotranspiration(ET) (mm)	186	146	181	138	173	130	173	135	550	542	728	155
Net Surplus (NS) (mm)	469	510	474	517	482	525	483	520	105	113	-73	500
Site Infiltration (mm)	0	155	0	178	0	183	0	160	1927	4756	4195	181
Runoff (mm)	469	354	474	339	482	343	483	360	22482	10838	15205	319
Underdrain (mm)	N.A		N.A		N.A		N.A		13644	9100	13599	36342
Surface Flow (mm)	N.A		N.A		N.A		N.A		8839	1739	1606	12184
INPUTS (Volumes)												
Precipitation (m^3)	2686	9105	4782	16965	1638	5568	1965	6812	459	197	197	50370
OUTPUTS (Volumes)												
Evapotranspiration (m^3)	762	2022	1323	3582	432	1102	518	1404	385	163	218	11917
Net Surplus (m^3)	1924	7082	3458	13383	1206	4466	1448	5408	74	34	-22	38452
Site Infiltration (m^3)	0	2159	0	4600	0	1554	0	1660	1349	1427	1259	13904
Runoff (m^3)[1]	1924	4923	3458	8783	1206	2912	1448	3748	15738	3251	4562	24549
Underdrain (m^3)[2]	N.A		N.A		N.A		N.A		9551	2730	4080	16360
Surface Flow (m^3)[2]	N.A		N.A		N.A		N.A		6187	522	482	7190

Table 4.4 is a summary of the PCSWMM modelling results for the post-development scenario that included the three BRCs. Infiltration into the ground include the pervious areas of the sub-catchments, as in Section 4.2.2.2, plus also the infiltration from the bottoms of the BRCs.

Adding in the BRCs into the development, increased the total infiltration to 13,904 m^3. However, this amount did not meet the objective of matching the pre-development infiltration of 28,766 m^3. Table 4.5 summarizes the performances of the BRCs in regard to water balance with respect to the whole site and also isolating the performance of the Davis BRCs.

Table 4.5: Summary of Water Balance Performance of Bioretention Cells

	Whole Site	**Davis Dr. BRCs**
Pre-development infiltration (m^3)	28766	8993
Post-development infiltration, no mitigation (m^3)	10228	3359
Shortfall in infiltration before mitigation (m^3)	18538	5634
Post-development infiltration, with BRC mitigation (m^3)	13904	5899
Percentage increase in infiltration from BRCs	36%	76%
Shortfall in infiltration after mitigation (m^3)	14862	3094
Percentage of shortfall of all required mitigation	80%	55%

From Table 4.5, it can be seen that the Davis BRCs performed better in isolation than all the BRCs of the site but still were only able to provide 48 % of the required mitigation for the shortfall in infiltration.

Table 4.6 provides a comparative summary of the results for the three BRCs. One of the most notable observations, is the amount of surface flow experienced by the Bathurst BRC, compared to the other BRCs. It appears that the amount of flow overwhelmed its capabilities to absorb and process stormwater runoff. Also, the Bathurst BRC had considerably less infiltration than the other BRCs, likely due to the small IWS, since there was a smaller offset to the underdrains of the Bathurst BRC than the Davis BRCs, due to the seasonally high groundwater table.

Table 4.6: Comparative Summary of the BRCs

Bioretention Cell	Total Inflow	Evaporation Loss	Infil Loss	Surface Outflow	Underdrain Outflow
Davis East	16649	520	4429	478	11222
Davis West	13011	430	4964	308	7309
Sub-Total	**29660**	**950**	**9393**	**786**	**18531**
Percentage	**100%**	**3.2%**	**31.7%**	**2.6%**	**62.5%**
Bathurst	21330	433	2044	5228	13625
Total	**50990**	**1383**	**11437**	**6014**	**32156**
Percentage	**100%**	**2.7%**	**22.4%**	**11.8%**	**63.1%**

Note, all numeric values, except percentages, are in m^3

4.2.3.4 Sensitivity Analyses for Continuous Precipitation

As presented in the previous section, adding in BRCs was not sufficient to meet the objective of matching pre-development infiltration conditions. However, one could ask what conditions would potentially allow that objective to be met?

In the relevant literature review section of this thesis, two aspects of BRC design that were determined to be influential in the performance of BRCs are the infiltration ability of the soil in the bottom of a BRC and the volume of the internal water storage (IWS) below the invert of the underdrain. These two aspects of BRCs were further evaluated in regard to water balance.

In Ontario, agencies have often cited 15 mm/hr as the critical value in determining whether or not infiltration can be considered applicable in the design of a BRC (TRCA and CVC, 2010). An infiltration rate of 15 mm/hr could represent a typical sandy silt material. In other words, generally, any soil that is more cohesive than a sandy silt should not be considered suitable for infiltration. Whereas, any soil that is less cohesive than a sandy silt, would be considered suitable for infiltration practices. As a result, this value of 15 mm/hr was used in one of our sensitivity analyses. The infiltration rate of 15 mm/hr was set in the LID Editor of PCSWMM for the bottom of each of the BRCs and the model was run again. Table 4.7 shows a summary of the results.

Table 4.7: Summary of Results of Sensitivity Analysis Changing Infiltration Rate

Description	Whole Site	Davis Dr. BRCs
Pre-development infiltration (m^3)	28766	8993
Post-development infiltration, no mitigation (m^3)	10228	3359
Shortfall in infiltration before mitigation (m^3)	18538	5634
Post-development infiltration, with BRC mitigation (m^3)	22878	8926
Percentage increase in infiltration	124%	165%
Shortfall in infiltration mitigation (m^3)	5888	67
Percentage of shortfall of all required mitigation	32%	1%

As can be seen in Table 4.7, the BRCs of the entire site were still not capable of fully mitigating the loss of infiltration due to urbanism; however, the Davis BRCs were almost able to completely mitigate that loss of infiltration (within 1%) for their sub-catchment areas.

The second sensitivity analysis dealt with the IWS of the BRCs. In this case, the volumes of the BRCs were arbitrarily doubled to see what change that would make in the performance of the BRCs. To do this, the offset of distance of the underdrains from the bottom of the BRCs was doubled in the LID Editor in PCSWMM. Prior to this sensitivity analysis, the underdrain offset distance of the Bathurst BRC was only 0.1 m and the average underdrain offset distance was 0.4 m for the Davis BRCs. The new underdrain offset distances became 0.2 m and 0.8 m for the Bathurst and Davis BRCs, respectively. In construction terms, this could be accomplished by digging the bottom of a BRC deeper and filling it with gravel. There would be the provision that a groundwater table or bedrock did not exist preventing the deeper excavation. In reality, the Bathurst BRC would not be able to accomplish this due to the seasonally high groundwater table. Table 4.8 is a summary of resulting runs in PCSWMM.

Table 4.8: Summary of Results of Sensitivity Analysis Changing Internal Water Storage

Description	Whole Site	Davis Dr. BRCs
Pre-development infiltration (m^3)	28766	8993
Post-development infiltration, no mitigation (m^3)	10228	3359
Shortfall in infiltration before mitigation (m^3)	18538	5634
Post-development infiltration, with BRC mitigation (m^3)	15426	7005
Percentage increase in infiltration	51%	108%
Shortfall in infiltration mitigation (m^3)	13340	1988
Percentage of shortfall of all required mitigation	72%	35%

As can be seen in Table 4.8 the BRCs were unable to fully meet the objective of mitigating the loss of infiltration. However, the amount of infiltration for the Davis BRCs increased by 108%, effectively more than doubling the amount of infiltration.

The above sensitivity analyses show that within the confines of this research, that although both the higher infiltration rate at the bottom of a BRC and the increased IWS of a BRC improve the performance of a BRC to mitigate the loss of infiltration due to urbanism, the infiltration rate at the bottom of the BRC is the more critical aspect.

4.2.3 Design Storm Simulation

4.2.3.1 Design Targets

As discussed in Section 3.6.1, Evaluation Criteria, besides water balance, erosion control from frequent rainfall events and peak flow control of more major storm events are required in stormwater management practices. In the previous section, continuous precipitation data over a long period of time was used to analyze water balance; however, for erosion control and peak flow rates, single events were modelled in PCSWMM.

For erosion control, according to local design criteria, the volume of runoff from a 25 mm design storm with a Chicago type distribution over a four-hour duration must be detained on-site and released over a 24-hour period (LSRCA, 2016). This criteria was used for part of the design of a stormwater management facility.

For pre-development peak flows of major storm events, attempts were made to model this in PCSWMM with imperviousness set to zero. However, the resulting flow values were deemed to be too low when compared to other methods. This may be since the underlying program in PCSWMM, which is SWMM, was created to simulate runoff primarily from urban areas (Rossman, 2015). As an alternative, to establish pre-development flows, the Rational Method was utilized (Bedient and Huber, 1988). This method has a long history in establishing peak flows in simple cases and is still considered a sufficiently accurate method of runoff estimation (Schwab et al, 1981). The method uses the following simple equation:

Equation 2: Rational Method

$$Q_p = 0.0028\ CiA$$

where $Q_{p=}$ peak flow rate (m^3/s)

C=runoff co-efficient, variable with land use and soil type (dimensionless)

i=rainfall intensity (mm/hr), chosen for a design return period and dependent of the time of concentration, t_c

$t_{c=}$time of concentration (min), time for rainfall to travel from the remote part to the outlet(min./hr)

A=watershed area (ha)

Appendix A contains details of this calculation. As a result of the calculations, the design target flows for the design of stormwater management facilities were determined, as listed in Table 4.9.

Table 4.9: Design Target Flow Rates for Peak Flow Control

Return Period	Peak Flow (m^3/s)
2	0.20
5	0.27
10	0.32
25	0.36
50	0.43
100	0.48

4.2.3.2 Post-Development, Without Stormwater Management

The next step was to run the PCSWMM model for the post-development scenario without any stormwater management. The input parameters for this model are those listed in Table 4.1, in Section 4.2.1. As discussed in Section 3.6.3, Rainfall, modelling was done for 25 mm, 2-year, 5-year, 10-year, 25-year, 50-year and 100-year events for both the Chicago 4-hour duration and SCS Type II 12-hour rainfall distributions. The results of the modelling are shown in Table 4.10.

Table 4.10: Peak Flow for the Post-Development Scenario, without Stormwater Management

Rainfall Event	Peak Flow (m^3/s), Chicago 4-hour Storm	Peak Flow (m^3/s), SCS Type II 12-hour Storm
25mm	0.53	0.40
2 year	0.92	0.84
5 year	1.38	1.19
10 year	1.73	1.46
25 year	2.12	1.76
50 year	2.60	2.04
100 year	2.82	2.19

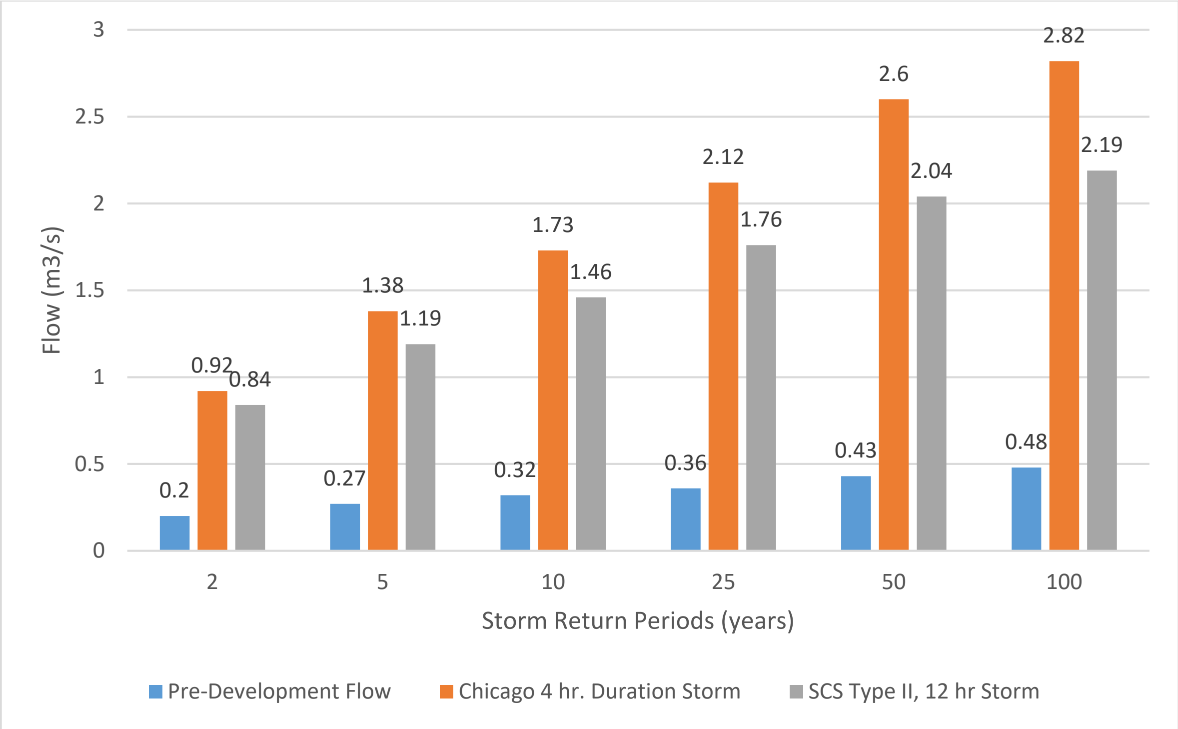

Figure 4.6: Bar Chart of Pre-Development Flows and Post Development Peak Flows, for Various Storm Types, Without Stormwater Management

As was expected, due to the higher rainfall intensities, the peak flows from the Chicago storms produced higher peak flows. They are more representative of summer convection storms that have shorter duration and higher rainfall intensities. This scenario illustrated one of the impacts of urbanization, whereby the greater imperviousness and the faster stormwater conveyance systems increase the peak flows to damaging values; for example, for the100 year Chicago storms the post-development peak flows were 5.9 times the pre-development peak flow, for the same return period.

4.2.3.3 Post-Development, with Stormwater Management but no Bioretention Cells

In this scenario attempts are made to mitigate the damaging effects of these higher flows by using typical stormwater management techniques. In most cases in suburban medium density developments, stormwater management ponds with permanent pools are constructed. These ponds are constructed such that the permanent pool is in place to provide water quality treatment and then storage is available on top of that permanent pool to detain volumes held back by control structures which limit flows to pre-development peak levels.

In this study the typical stormwater management pond was substituted by oil/grit separator and an underground storage tank. This substitution was done for several reasons. Besides the economic reason discussed in Section 3.7, there are technical reasons. The SWMPD manual says that the tributary area to a SWM pond must be at least 5 ha. to capture enough water to sustain a permanent pool of water. The manual further recommends a tributary area greater than 10 ha is preferred, for the same reason. The combined areas of the sub-catchments in this study is 7.69 ha.

A concrete box storage tank with a bottom orifice, an orifice midway to the top and a weir at the top of the structure was conceptualized to contain the required volume. The design process was a trial-and-error process to ensure the design targets of Section 4.2.3.1 were being met. First trials attempted to make the bottom orifice small enough to control the flow from the bottom orifice of the structure such that the volume from a 25 mm storm of a Chicago 4-hour duration was released slowly over 24 hours. The volume from such a storm was modelled to be 1160 m^3. When dividing this volume by 24 hours, the resulting average flow is 0.013 m^3/s. The final critical design criteria for total volume were ensuring that the flows from the 100-year SCS Type II 12-hour storm did not exceed the 100-year pre-development flow. The SCS storm was found to be more critical than the Chicago storm because it produces more volume of water. This may be seen contrary to what one would consider since the Chicago storm produced higher post-development flows when modelled without stormwater management.

The final dimensions of the storage tank were 36 m by 40 m in horizontal dimensions by 2.3 m in height and had a total volume of 3312 m^3, not including freeboard. The summary of the control features is listed in Table 4.11 and the resulting flows and volumes are listed in Table 4.12. The designed tank has a slightly higher volume than 3,279 m^3 for the 100-year SCS storm.

Table 4.11: Dimensions of Control Features of the Storage Tank, without BRCs

Control Feature	Width (m)	Height (m)	Offset from Bottom (m)
Lower Orifice	0.1	0.075	0
Upper Orifice	0.35	0.15	0.8
Weir at Top	0.18	0.6	1.7

Table 4.12: Flows and Volumes for Post-Development Scenario with SWM but no BRCs

Storm	Pre-Development Flow (m^3/s)	Chicago 4-Hour Storm		SCS Type II 12-Hour	
		Flow (m^3/s)[1]	Volume (m^3)	Flow (m^3/s)1	Volume (m^3)
25 mm	0.013	0.013	1139	0.012	928
2 yr.	0.20	0.087	1429	0.101	1555
5 yr,	0.27	0.133	1902	0.140	2082
10 yr.	0.32	0.158	2255	0.182	2533
25 yr.	0.36	0.200	2601	0.286	2863
50 yr.	0.43	0.319	2945	0.406	3136
100 yr.	0.48	0.401	3125	0.478	3279

1. Flows are peak flows except for 25 mm event in which case flows are average flows, as per design criteria

4.2.3.4 Post-Development, with Stormwater Management and with Bioretention Cells

The next step was adding the three bioretention cells to the previous model. This was done the same way as in Section 4.2.2.3 for continuous modelling. The BRCs were added as additional sub-catchments that consisted of 100% as BRCs. That way all the runoff from the other upstream sub-catchments was clearly followed. When the past assortment of storms was modelled and the volumes listed in Table 4.12 were compared with the results, it became evident that the BRCs were contributing to volume control. For example, for the previously mentioned critical storm, the SCS 100-year storm, the model showed a post-development flow of 0.40 m^3/s, as compared to the pre-development flow of

0.48 m^3/s. Also, only 3124 m^3 of storage was consumed in the storage tank, as compared to 3,279 m^3 without BRCs.

As a result, there was an opportunity to reduce the size of the storage facility and modify the control features of the SWM facility. Again, a trial-and-error design process was involved, and the storage facility was modified to account for the contribution that the BRCs made to quantity control of post-development flows. The size of the storage facility was reduced to dimensions of 32 m by 40 m in horizontal dimensions with the same vertical height of 2.3 m. The resulting full volume was reduced from 3312 m^3 to 2944 m^3. Also, some of the orifice and weir dimensions were slightly modified, again by the trial-and-error design method to maximize SWM facility dimensions to effectively size the SWM facility to limit peak flows to pre-development values. The final resulting flows and respective volumes are represented in Table 4.13.

Table 4.13: Flows and Volumes for Post-Development Scenario with SWM and BRCs

Storm	**Pre-Development Flow (m^3/s)**	**Chicago 4-Hour Storm**		**SCS Type II 12-Hour**	
		Flow (m^3/s)	**Volume (m^3)**	**Flow (m^3/s)**	**Volume (m^3)**
25 mm	0.013	0.013	510	0.012	385
2 yr.	0.20	0.048	1093	0.076	1168
5 yr,	0.27	0.117	1462	0.136	1651
10 yr.	0.32	0.150	1803	0.173	2102
25 yr.	0.36	0.179	2178	0.272	2497
50 yr.	0.43	0.302	2571	0.401	2782
100 yr.	0.48	0.390	2759	0.481	2930

4.2.3.5 Sensitivity Analysis for Storm Events

As was done in continuous precipitation modeling, sensitivity analyses were done by varying the infiltration rate at the bottom of BRCs to 15 mm/hr and also by doubling the IWS. In the case of changing the infiltration rate, there was a noticeable difference in the required storage when the most critical storm, the SCS 100-year storm, was modelled. The resulting volume in the storage tank was reduced from 3279 m^3 to 3087 m^3. The peak flow was also reduced from 0.48 m^3/s to 0.38 m^3/s. Therefore, there was an opportunity to increase some of the dimensions of the outlets to let more flow out of the storage structure, thereby reducing the amount of required storage. Again, a trial-and-error design process was necessary to maximize the efficiency of the outlets but ensure that peak flows for all storm events still did not exceed pre-development flows. After modifications were made, the tank size was appropriately reduced and the required storage for that critical SCS 100-year storm was 2677 m^3.

The same type of exercise was performed with the doubling of volumes of the IWS. The offset distance from the bottom of the BRCs to the inverts of the underdrains was made twice as large and the critical SCS 100-year storm was again modelled. The resulting volume in the storage tank was reduced from 3279 m^3 to 3123 m^3 and the peak flow was reduced from 0.48 m^3/s to 0.40 m^3/s. These results were less than those from the previous exercise. Therefore, doubling the IWS did not have as much impact on reduction of required storage as increasing the infiltration rate of the soil in the bottom of the BRCs to 15 mm/hr.

4.2.4 Hydrologic Performance Discussion

4.2.4.1 Water Balance

From site reconnaissance, it was observed that the Davis BRCs were performing well by the fact that their surfaces were free of standing water after a typical rainfall event; whereas the Bathurst BRC was not performing as well, as indicated by the growth of wetland vegetation (as described in Section 4.1.1). The Bathurst BRC did not appear to do as well due to a potential combination of the following: excessive drainage area in comparison of the area of the BRC; relatively small IWS, sediment on top of mulch (post construction cleaning had still not been done at that time) and/or potential groundwater contribution. Computer modelling of continuous precipitation data from April 1 to November 30, 2018, was discretized to analyze the whole site and also isolate the Davis BRCs since they were deemed to be performing closer to expectation.

The purpose of the continuous precipitation modelling was to analyze the contribution BRCs were making towards water balance objectives. Although the Davis BRCs performed better than the Bathurst BRC, they were not able to fully mitigate the loss of infiltration due to the increase of imperviousness from urbanization. Approximately only 45% of the loss of infiltration was recovered in the scenario which isolated the Davis BRCs contribution to infiltration. The likely reason for this shortfall is due to the cohesiveness of the primarily silty clay soil. This hypothesis is supported by the monitoring of the drawdown of water levels in the BRCs.

The above conjecture was supported when the first sensitivity analysis was done. In that sensitivity analysis, the in-situ infiltration rate at the bottom of the BRCs was replaced with an infiltration rate indicative of a less cohesive soil, for example, a sandy silt material. When this change was made and the system remodelled, the Davis BRCs allowed the infrastructure to almost completely mitigate the loss of infiltration due to urbanism. The second sensitivity analysis of doubling the IWS of the BRCs did not prove to be as effective as the first sensitivity analysis.

4.2.4.2 Peak Flow Control Storage Reduction

Table 4.14 is a table summarizing the volumes that were required to be stored in the storage facility to reduce the peak flows of post-development below the peak flows of pre-development for both types of design storms and for all the required return periods.

Table 4.14: Summary of Volume (m^3) Required to Control Peak Flow

Return Period	SWM Only		SWM and BRCs		Percentage Reduction		
	Chicago	SCS (Type II)	Chicago	SCS (Type II)	Chicago	SCS	Average
2 year	1429	1555	1093	1168	24%	25%	24%
5 year	1902	2082	1462	1651	23%	21%	22%
10 year	2255	2533	1803	2102	20%	17%	19%
25 year	2601	2863	2178	2497	16%	13%	15%
50 year	2945	3136	2571	2782	13%	11%	12%
100 year	3125	3279	2759	2930	12%	11%	11%

It is interesting to note that although the BRCs did not have a favourable bottom infiltration rate, they were still able to reduce the required amount of volume of the SWM storage facility to control peak flows. The percentage of reduction was greater for more frequent storms of less volume as compared to less frequent storms of greater volume. This observation of the percentage of volume reduction being inversely proportional to the total volume is consistent with some past literature (Willard et al, 2017). This is likely since the available storage that can be captured by a BRC is finite whereas the volumes of the storms varied. The greater the ratio of BRC volume to storm rainfall volume, the greater the contribution to peak flow control the BRC can make. In terms of construction cost savings, the SCS 100-year storm was the most critical storm event. In this case, a SWM storage facility could be constructed 11% smaller with the presence of the BRCs than without BRCs, despite the fact the BRCs were constructed in low permeability soils.

Again, a sensitivity analysis was completed, assuming less cohesive soils that exhibited infiltration rate of 15 mm/hr. In this case, the require volume in the storage facility was 2677 m^3 for the critical SCS 100-year storm. This value represented an increase in the reduction percentage to 18% of required volume to control peak flow of the critical SCS 100-year storm design event.

By increasing the infiltration rate to 15 mm/hr, the storage from the traditional SWM only scenario was reduced by 602 m^3 (3279 m^3 less 2677 m^3). Recall that the volume of runoff of the 25 mm event was 1160 m^3. Therefore, if a reduction credit in required storage is given equivalent to the volume of runoff from a 25 mm event, as in the case of Appendix B of the LSRCA Technical Guidelines for Stormwater Management (LSRCA, 2016), that would result in a storage tank being undersized by 558 m^3.

The likely reason that BRC could not utilize a more favourable bottom infiltration rate is due to the infiltration rate of the BRC's upper filter media not being high enough for that intense stormwater flowrate and amount of volume of runoff from the sub-catchment areas. In such an event, that flow that would exceed the infiltration rate of the BRC would pass through the upper bowl of the BRC as surface runoff. The PCSWMM modelling for the SCS 100-year event calculated a total runoff of 15,057 m^3 of runoff from the sub-catchment areas and 11,298 m^3 of that (75%) was surface outflow.

4.2.4.3 Peak Flow Delay

One aspect of stormwater management that was deemed worth analyzing was timing of peak flows. This aspect can sometimes be important because if timing of peak flows from various sub-catchments coincide, their peak flows are compounded; however, if some peaks are delayed so that they do not coincide, then the compounding aspect is minimized. This aspect is important for major storm events, such as a 100-year storm. In Figure 4.6, the outflow hydrographs of the post-development with SWM scenario without BRCs and the post-development with SWM and BRCs scenario are plotted. In the scenario with BRCs, the time of the peak flow is delayed by approximately 7 minutes when comparing the output files in the appendices. This is not considered an appreciable difference since it is within the time span of rainfall time steps. It should be noted that for the scenario with BRCs, 77% of the inflow resulted in surface outflow. Time lag can occur for smaller events but that would be of little consequence if the issue is flooding during major events.

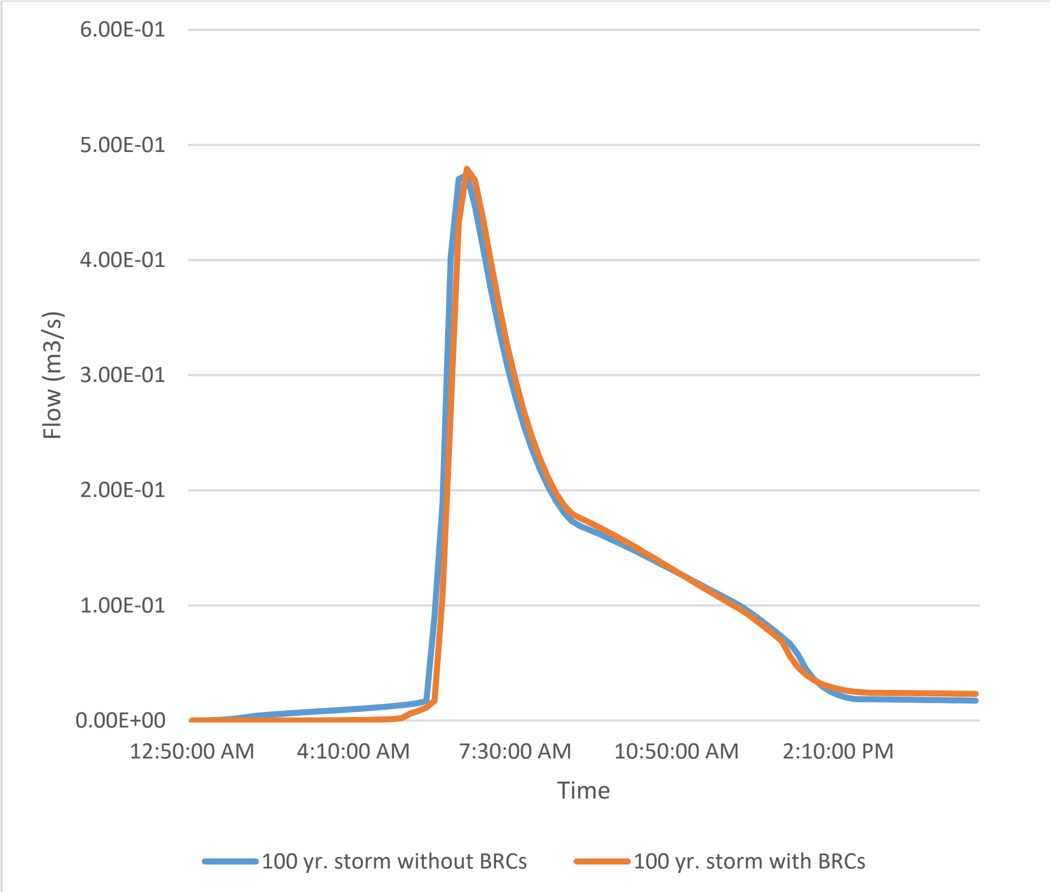

Figure 4.7: Hydrograph plots of SCS 100-year storm without BRCs and with BRCs

4.2.4.4 Volume Control

In the LSRCA Technical Guidelines for SWM, a target in volume control is for runoff from a 25 mm rainfall event to be totally retained/treated on site. This target will help reach the objective of minimizing phosphorus loading on watercourses draining to Lake Simcoe which is a main goal of the Lake Simcoe Protection Act (LSRCA, 2016). However, if this target cannot be met, LSRCA have alternative approaches that include milestones of also 5 mm and 12.5 mm rainfall events (LSRCA, 2016).. To analyze how BRCs can assist towards these milestones, the post-development model with SWM and BRCs was modelled with rainfall events of 5 mm, 12.5 mm and 25 mm SCS distributions. Figures 4.8, 4.9 and 4.10 show the results of these modelling exercises along with respective rainfall hyetographs.

In a statistical analysis from 1960 to 1992 of rainfall data at Toronto Pearson International Airport, it was determined that the average rainfall was approximately 5 mm ((Behera et al, 1999). For the 5 mm event, there was no flow exiting the BRCs by way of underdrain or surface flow for the Davis BRCs and minimal underdrain flow only in the Bathurst BRC, peaking at 0.002 m^3/s. There was underdrain flow from all three BRCs for the 12.5 mm event, as shown in the figure for that event but no surface outflow. This is an important aspect to appreciate since although that flow that exited through the underdrains was not totally retained on the site, it did receive some filtration treatment via the media in a BRC. For the 25 mm event, there was underdrain and surface outflow from all three BRCs with the Bathurst BRC showing an appreciable greater flow, as represented by the yellow line in Figure 4.10.

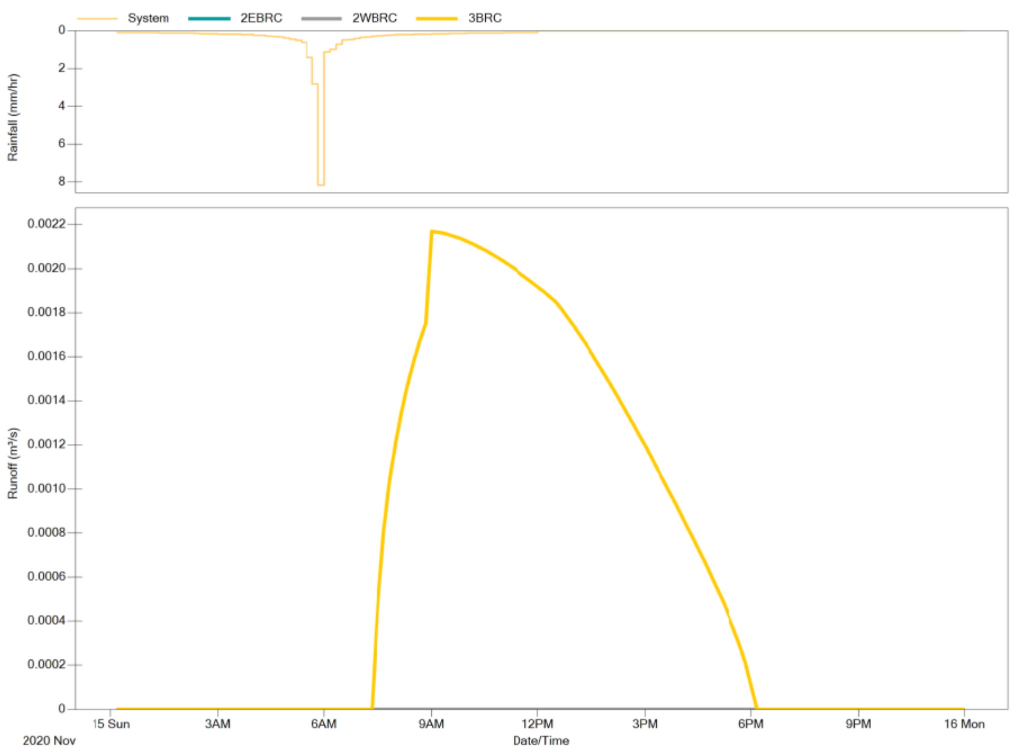

Figure 4.8: 5 mm, SCS (Type II) 12 Hour Rainfall Event

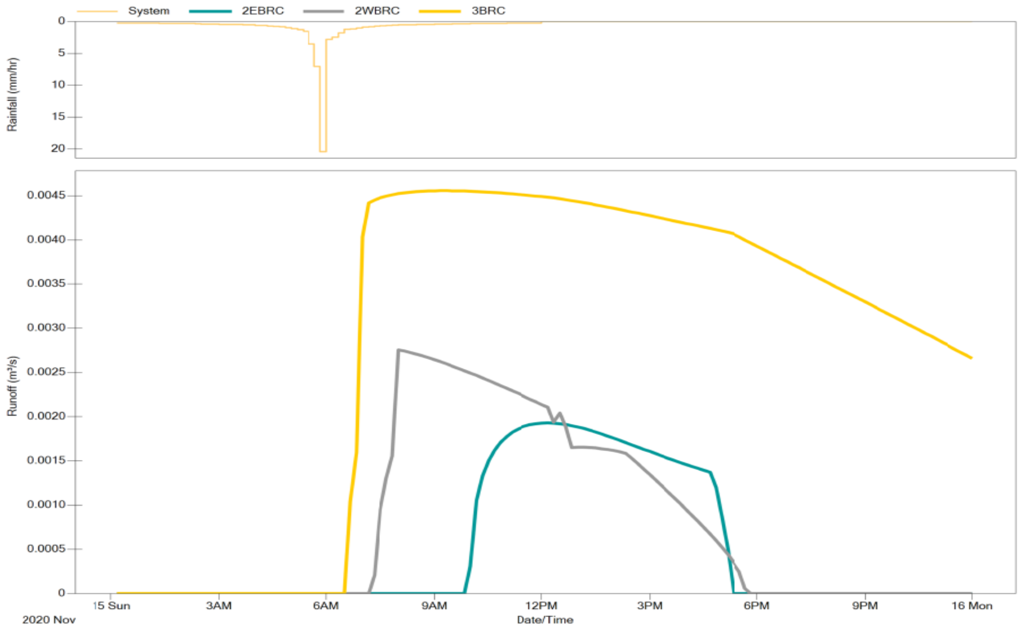

Figure 4.9: 12.5 mm, SCS (Type II) 12 Hour Rainfall Event

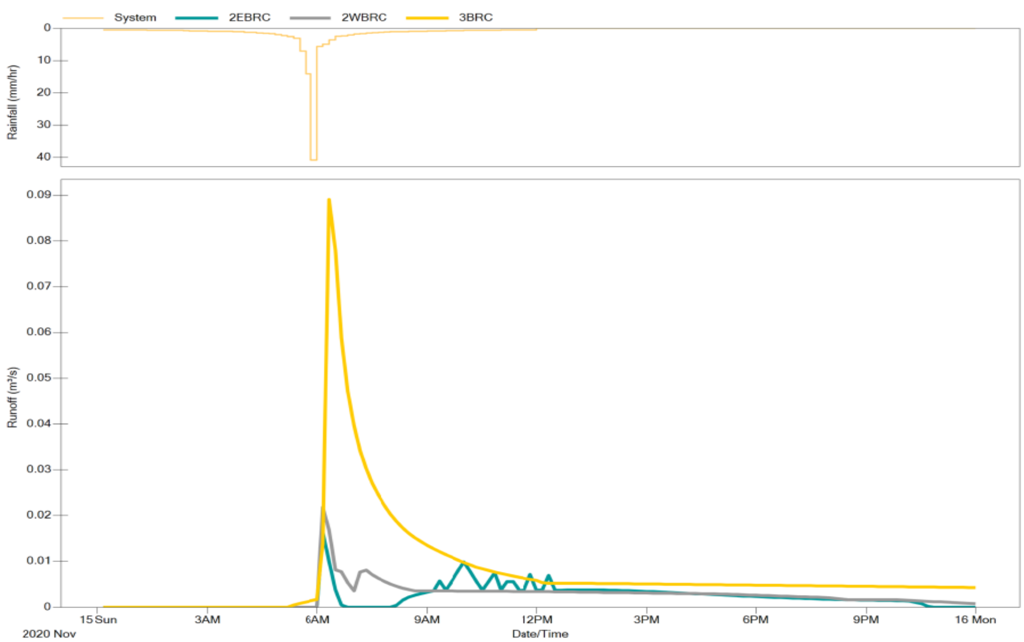

Figure 4.10: 25 mm, SCS (Type II) 12 Hour Rainfall Event

4.3 Cost Examination

4.3.1 Unit Costs and Cost Estimates

As mentioned in the Cost Examination methodology section, Section 3.7, cost examination was done for capital costs and maintenance costs for a traditional SWM approach and for the LID approach using BRCs.

For the traditional SWM system, *Jellyfish* units were considered for water quality treatment, as discussed in Section 3.7. To get sizing, capital costs and maintenance requirements, the manufacturer, Imbrium, was contacted. It should be noted that one of the four *Jellyfish* units that would be required, Model No. JF6-5-1, is the same model noted in the sample application, MECP ECA Number 3221-ACBLZK, discussed in Section 3.7 (MECP, 2016). For the tributary area to the Bathurst BRC, two *Jellyfish* units were required, one for each of sub-catchments 3S and 3E, draining to Bathurst BRC, due to the large overall drainage area to the Bathurst BRC. Detailed cost calculations are provided in Appendix C.

The Jellyfish units are designed to be off-line and require the cost of additional manholes for the necessary configuration. The Mosaik Glenway tender was referenced for typical manhole costs. The final capital cost for the traditional SWM quality control system is $ 577,403, including 25% for engineering and contingencies.

For the storage tank, the tender for Mosaik Glenway was consulted. At the site, a concrete tank made of prefabricated sections with a volume of 2495 m^3 cost $ 1,016,440 to construct. For this research, this cost was pro-rated based on volume. The manufacturer for the storage tank, Storm Trap^{TM}, was contacted and a representative of that firm confirmed that that was a reasonable approach to deriving a cost estimate for a study of this type. For the traditional SWM system, a tank holding 3312 m^3 was required. The pro-rating of the tank resulted in a capital cost of $ 1,349,278. However, the tender was completed in 2014 dollars and had to be updated to reflect 2020 dollars, to be comparable to the costs of the *Jellyfish* units. For that component, Statistics Canada website was used and an indexing factor of 1.199 was determined to bring 2014 costs up to 2020 costs. After indexing was applied and 25% was added for engineering and contingencies, the final tank cost was calculated to be \$ 2,022,231.

Regarding maintenance costs for the *Jellyfish*, information was obtained from the manufacturer. The units require quarterly inspections, including annual washing of the cartridges and replacement of the cartridges every four years. For maintenance, a life cycle of fifty years was chosen to be consistent with other similar analyses (STEP, 2013). For over the 50-year life span, maintenance costs for the traditional SWM water quality system totalled \$ 386,600. Another \$19,500 was added for maintenance costs of the storage tank.

For the LID SWM system that included BRCs, the water quality treatment costs of the *Jellyfish* units were replaced with the costs of the BRCs. The costs of the BRCs and appurtenances, such as ditch inlet catch basins, were extracted from the tender for a total of \$ 205,086. Again, indexing had to be applied to the cost to bring these tendered costs to 2020 dollars. The final capital costs of BRCs and appurtenances totalled \$ 307,373, including 25% for engineering and contingencies.

For the storage tank, pro-rating and indexing were again applied to the tank that was constructed at the Mosaik Glenway site. In this case, however, the required volume of the tank was reduced by 11% to 2944 m^3 as a result of the influence of the BRCs. The resulting cost was \$ 1,797,538, after adding 25% for engineering and contingencies.

With respect to maintenance for the BRCs, the STEP Life Cycle report was consulted (STEP, 2013). Costs were derived for litter cleaning and other regular maintenance tasks plus replacement of filter media every ten years. This filter media replacement every ten years appears reasonable from observations made by STEP for older BRCs that they have been monitoring for the last decade (STEP, 2019). After applying the same maintenance costs for the storage tank as in the traditional SWM solution, the total maintenance costs of the SWM system with BRCs totalled \$ 416,037 over the fifty-year period.

4.3.2 Summary of Cost Estimates and Comparative Analysis

Table 4.15 provides a summary table of the cost estimates provided in the previous section.

Table 4.15: Summary of Cost Estimates

	Traditional SWM System		**LID SWM System with BRCs**	
Capital Costs	**Unit Type**	**Cost**	**Unit Type**	**Cost**
Water Quality	Jellyfish	$ 577,403	BRCs	$ 307,373
Water Quantity	Storage Tank	$ 2,022,231	Storage Tank	$ 1,797,538
Sub-Total		**$ 2,599,634**		**$ 2,104,911**
Maintenance Costs				
Water Quality	Jellyfish	$ 386,600	BRCs	$ 396,537
Water Quantity	Storage Tank	$ 19,500	Storage Tank	$ 19,500
Sub-Total		**$ 406,100**		**$ 416,037**
Total		**$3,005,734**		**$ 2,520,948**

As can be seen in Table 4.15, the capital costs of both the water quality component and water quantity component were less for the LID system with BRCs than for the traditional SWM system. Cost of a storage tank was expected to be less for the LID system since the required volume is 11% less due to the contributions of the BRCs. The fact that the costs of the BRCs were found to be less than the *Jellyfish* oil/grit separators was unexpected but those were the costs that were determined in this study. Overall, the LID SWM system capital cost $ 494,723 or 19% less than the traditional SWM system.

With respect to maintenance, the costs wee similar for both systems. Prior to the study, it was thought that BRCs may require more maintenance costs due to exposure to wind-blown litter and potential variances with plant growth. The underground *Jellyfish* oil/grit separators would not be subject to wind-blown surface litter and would not have the variances of plant growth to deal with. However, following the manufacturers recommended maintenance proved to be more expensive than expected for the *Jellyfish* oil/grit separators and BRCs appear to require less maintenance than expected, as per studies by STEP (STEP, 2019).

Since maintenance costs for both systems are similar, saving of capital costs carried through to the final sum and kept the LID system with BRCs $ 484,786 less than the overall cost of the traditional SWM system.

Chapter 5 – Conclusions and Recommendations

The objective of this research was to use results of monitoring and computer modelling of a recently constructed low density residential subdivision in the Greater Toronto Area to better understand the contribution that BRCs make to quantity control in stormwater management. Besides technical efficiencies, cost efficiencies were also evaluated.

5.1 Summary of Hydrological Aspects

Initially, different aspects of the three BRCs were highlighted. From a site reconnaissance, it was observed that one of the BRCs was not performing as expected in that standing water was observed days after a precipitation event and wetland type vegetation was establishing itself. This BRC, the Bathurst BRC, even though it was the largest in surface area of the three BRCs, was servicing a drainage area beyond its capacity, particularly regarding directly connected impervious area. Computer modelling confirmed the challenging conditions that this BRC was set in. As a result, the conclusions drawn from this study are more in concert with the performance of the other two BRCs, Davis East BRC and Davis West BRC.

With the use of monitoring data, in-situ infiltration rates were estimated for the bottoms of the BRCs. Using these infiltration rates, modelling using continuous rainfall data that corresponded to the monitoring period, water balance calculations were performed. It was determined that the BRCs were not totally successful, by themselves, to mitigate the entire loss of infiltration due to increased imperviousness from urbanization. In this study, they were able to recover approximately 45% of the loss of infiltration. This type of observation is consistent with other past studies of BRCs (Li et al, 2009).

Agencies in Ontario often cite an infiltration rate of 15 mm/hr as the critical value in determining whether or not infiltration can be considered when designing a BRC (TRCA and CVC, 2010). An infiltration rate of 15 mm/hr could represent a typical sandy silt soil. When this value of 15 mm/hr was used in a sensitivity analysis, the BRCs were found to be almost successful (within 1%) in mitigating that loss of infiltration.

Another sensitivity analysis raised the offset of the underdrain to double the size of the IWS. This second sensitivity analysis improved the mitigation of the loss of infiltration but not to the same extent that changing the infiltration rate at the bottom of the BRCs to 15 mm/hr. However, when the IWS was doubled in BRCs, the amount of infiltration also doubled, despite the native soil conditions being less than ideal. To conclude, native infiltration rate is more important than IWS size, but when native infiltration rate is low, a large IWS should be designed.

Regarding design storm events, BRCs reduced the required storage for 2 to 100-year events from 24% to 11%, respectively. The 11% reduction of volume is the most important value since it represents the 100-year storm which is what the storage tank would ultimately have to be designed for and that is the scenario where a potential saving in capital costs can be analyzed. This is a positive result considering the cohesive nature of the soils of the site. Therefore, the fact that a site has a soil that does not have an infiltration greater or equal to 15 mm/hr should not be eliminated from the opportunity of receiving a credit for reduction of volume necessary to fulfill objectives of a SWM system, as is the case in one of the cited design guidelines (LSRCA, 2016). The results of this research can be used to inform agencies, such as TRCA, CVC and LSRCA, when considering future design guidelines of BRCs.

As in the continuous modeling case, a sensitivity analysis was done using a soil with a favourable infiltration rate of 15 mm/hr. The cited design guidelines can provide a volume credit of the volume of a 25 mm runoff event, if several conditions are met, including the soil having an infiltration rate of 15 mm/hr or better (LSRCA, 2016). This study showed by modeling at the said infiltration rate that the reduction of storage is far less than the volume of a 25 mm runoff event. This is likely due to the infiltration rate of the BRC’s upper media not being high enough for that flowrate and volume of runoff to be absorbed by the BRC. In such an event, that flow that exceeded the infiltration rate of the BRC’s upper filter media would pass through the upper bowl of the BRC as surface outflow, as shown by the PCSWMM modelling.

Other hydrological aspects that were studied included time of concentration of peak flow and volume control of small design events of 5 mm, 12.5 mm and 25 mm. For time of concentration of peak flow, no appreciable difference was noted from PCSWMM modelling of peak flow hydrographs with or without BRCs for large storm events, like a 100-year storm. For the 5mm design events, except the Bathurst BRC, BRCs were found to be able to absorb the runoff from such an event without any downstream flow. For the 12.5 mm event, there was underdrain flow from all BRCs but there was no by-passing of the BRCs via surface outflow; and therefore, runoff received filtration treatment from the BRCs. During the 25 mm event, all BRCs experienced surface outflow so the BRCs were not able to fully treat 25 mm storm events.

5.2 Cost Implications

Any responsible engineering analysis should include costs, when feasible, and such was done in this study from both capital cost and maintenance cost perspectives. Comparison was made between a traditional SWM design that would have approved oil/grit separators for water quality treatment and storage tank for water quantity control versus an LID design that had BRCs for water quality treatment and a storage tank for water quantity control. From a capital cost point of view, the traditional SWM design was more expensive than BRCs for water quality treatment. For water quantity control, the LID based system was 11% less expensive since the storage tank did not need to be as large. Overall, capital costs for the LID based system was approximately $ 494,723, or 19%, less expensive. Maintenance costs over an assumed 50-year lifespan where comparable. Overall, the LID based system was $ 484,786, or 16% less expensive. Given the overall hydrological benefits discussed above and the favourable cost implications makes the LID system a highly valued engineering solution.

5.3 Design Recommendations

One of the most notable outcomes of the study was a way to help decide on an appropriately sized BRC. From the notable difference in outcomes between Bathurst and Davis BRCs, it is felt that consideration be given to the ratio of directly connected impervious area and the volume of water the BRC could hold. It is felt that this may be a more appropriate measure than just comparing the directly connected impervious area with the area of the BRC. From this study, a maximum ratio of 15:1 or less of directly connected impervious area to volume of the BRC may be a criterion for approval agencies, such as TRCA, CVC and LSRCA, to consider.

If BRCs are constructed in less-than-ideal soil conditions, the larger the IWS, the greater the amount of infiltration. Underdrain offset distances of 0.5 m or greater from the bottom of the BRC's gravel bed to the invert of the underdrain should be considered.

Another recommendation to design may be considered more of a policy change. In that even if the soils are not favourable from an infiltration perspective, consideration should be given for a volume credit that BRCs can provide that may allow storage facilities for major storm events be sized recognizing the contribution that BRCs can make. This research only reviewed one constructed case. More constructed cases should be reviewed to make more solid recommendations on volume credits, or potentially, such analyses may have to be done on a case-by-case basis. This study has shown that giving a volume credit equivalent to the volume of a 25 mm event may not be the most appropriate approach to quantifying that credit (LSRCA, 2016).

5.4 Recommendations for Future Study

In this study, BRCs were successfully analyzed within the objectives of the thesis; however, in the process some recommendations for future study were realized.

Although some monitoring data was successfully utilized, the monitoring program was not designed with the objectives of this study in mind. A similar study with monitoring specifically designed to assist such a study would be beneficial. For example, actual measurements of any of the following would be helpful in better understanding the dynamics of BRCs: surface runoff into a BRC, flow through underdrains and surface overflows. Such data could also assist in any potential calibration of a hydrologic model.

Regarding modelling, it is recommended to repeat a similar study using an equivalent model to PCSWMM. Each computer simulation model has its own peculiarities in how it models certain hydrologic and hydraulic processes. For example, underdrains are modelled using a simplified approach in PCSWMM which may, or may not, affect the results. Monitoring data mentioned above may highlight underdrain flow characteristics in a BRC. Another issue of PCSWMM modelling was noted when dealing with the Bathurst BRC. In reality, it receives piped flow but PCSWMM has no means of simulating piped flow into a BRC the way piped flow can enter into a SWM pond. PCSWMM always assumes that a BRC receives surface flow. Modelling in this study had to make the same assumption.

Another software which could be considered is Visual OTTHYMO (VO) (Smart City, 2020). An older version of this program was used in the design of the SWM plan for the Mosaik Glenway subdivision, but it did not have any routines to model BRCs at the time. However, VO now has such routines. For example, VO has a different approach to directly and indirectly connected imperviousness than PCSWMM and it would be interesting to see if it would produce similar results.

Finally, infiltration at the bottom of a BRC appears to have a significant role in determining the performance of a BRC. When comparing the monitoring data of water levels and how they actually decreased exponentially, Figure 4.1, contradicted the straight-line approximation modelled in PCSWMM, Figures 4.4 and 4.5. Expanding this study by incorporating more sophisticated modelling of the vadose zone flow, such as HYDRUS-1D, or even HYDRUS-2D (PC Progress, 2017), which utilizes Richards equation to mathematically model the physical process of flow in the vadose zone, would be of benefit.

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